CHAPTER 14

PUMP STATIONS

Chapter 14 - Pump Stations
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14.1 Overview

14.1.1 Introduction

Stormwater pump stations are used to remove stormwater for highway sections that cannot be drained by gravity. Because of their high construction and operational cost and potential cost of malfunctions, their use should be limited to where no other system is feasible. Stormwater pump stations generally have high short-term capacity requirements and infrequent use, making them very costly on a per-use basis. The cost of a gravity system must be compared with the life-cycle costs (construction, maintenance and operation) of the pumping station. A pumping station should only be used when the life-cycle cost of the pump station is demonstratively less than the alternative gravity system.

The hydraulic analysis of a pump station involves the interrelationship of 3 components:
- the inflow hydrograph,
- the storage capacity of the wet well and the outside storage, and
- the discharge rate of the pumping system.

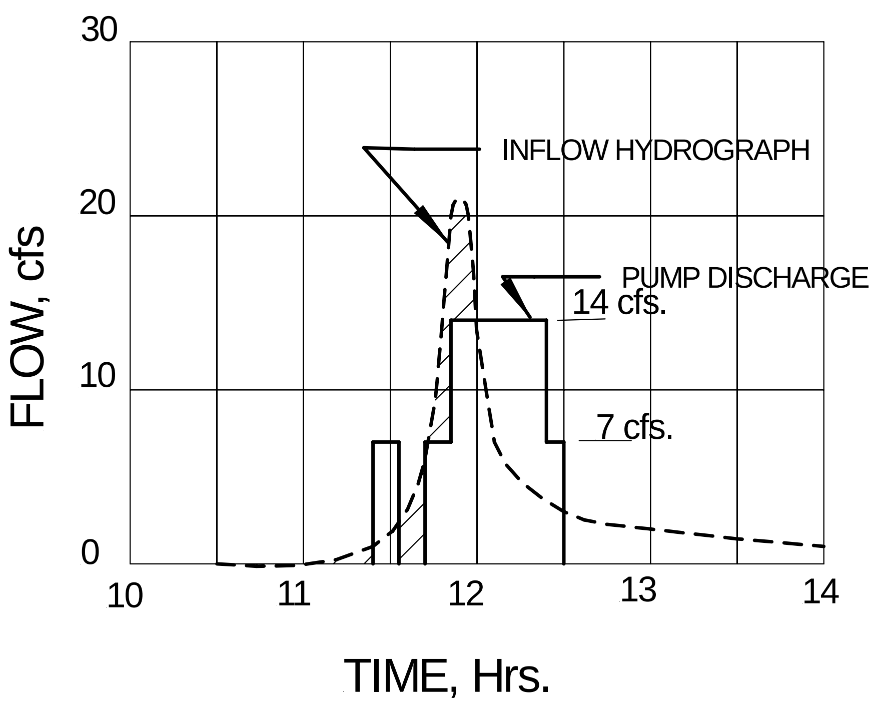


Fig 14-1 Pump Station Operation Hydrographs

The inflow hydrograph is determined by the physical factors of the watershed and regional climatological factors. The discharge rate of the pump station is often controlled by outfall capacity. Therefore, the main objective in pump station design is to store enough inflow (volume of water under the inflow hydrograph) to allow station discharge to meet specified limits while not exceeding the freeboard requirements at the inlets. A minimum volume of storage is required to prevent excessive pump cycling. Even where there are no limitations to pump station discharge, a design that balances storage and pumping capacity provides the most economical design since storage permits use of smaller and/or fewer pumps.

14.1 Overview (continued)

14.1.2 Design Choices

There are many choices to be made regarding pump stations for management of storm water along highways; some are driven by topography and the interaction of collection and discharge systems, others are related to construction and operating costs. Presented below are some of the areas to be considered.

Location/Building Design

- Site Access
- Site and Building Lighting
- Site and Building Security
- Equipment Access
  - Roof Hatches
  - Monorails
- Safe Workspace Environment
  - Potable water supply
  - Ventilation
  - Confined Space Hazard sensors

Operational/Mechanical Design

- Type: Wet pit vs. Dry Pit.
- Pumps:
  - Power & Back-up Systems
  - Type
  - Drive
  - Capacity and Number
  - Operational Size and Volume Requirements
    - Submergence Depth
    - Side Clearances
  - Cycling Plan
    - Hours, Number of Starts
  - Solids Handling Capacity
- Operation Plan/Storage
- Operating Monitoring Controls
  - Automatic and Manual Operation
  - Backup Systems
  - Non-operation Signals
  - Water-level Sensors
  - Hour meters
  - Number-of-starts meters

14.1 Overview (continued)

Operational/Mechanical Design (continued)

- Intake System
  - Trash Racks
  - Inlet type

- Discharge: Force Main vs. Gravity
  - Flap Gates
  - Valves

Many of the decisions regarding the above are currently based on engineering judgment and experience. To assure cost-effectiveness, the designer should assess each choice and develop economic comparisons of alternatives on the basis of annual cost. However, some general recommendations can be made which will help minimize the design effort and the cost of these expensive drainage facilities. These are discussed in the following pages of this chapter.

In the development of the Valley Freeway System in the Phoenix area, ADOT has adopted/standardized a design approach applicable to urban freeway pump stations. The station is based on a peak pump capacity of 200 cfs using 4-50 cfs (4-22,500 gpm) pumps. The elements of this standardized design will be presented as appropriate. Discussions with ADOT maintenance personnel should be had early in the design process.

For further information on the design and use of pump stations, see *Highway Storm Water Pump Station Design Manual*, FHWA-HEC-24 and the other references given at the end of this chapter. The Hydraulic Institute, 9 Sylvan Way, Parsippany, New Jersey, 07054-3802 has developed standards for pumps. Pump station design should be consistent with these standards.

14.2 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in the technical publications.

Table 14-1 Symbols And Definitions

Symbol	Definition	Units
A	Minimum distance from back wall to trash rack	ft
B	Maximum distance between a pump and the backwall	ft
C	Average distance from floor to pump intake	ft
D	Pump diameter	in.
DHW	Design high water elevation	ft
H_f	Friction head	ft
H_l	Losses through fittings, valves, etc.	ft
H_s	Maximum static head	ft
H_t	Storage depth	ft
H_v	Velocity head	ft
H_x	Depth for storage volume	ft
INV	Inlet invert elevation	ft
L	Length of wet well	ft
N	Number of equal size pumps	-
NPSH	Net Positive Suction Head	ft
Q_i	Inflow	ft^3/sec
Q_p	Total capacity of all pumps Peak Discharge Rate	ft^3/sec
S	Minimum submergence at the intake	ft
t_c	Minimum allowable cycle time	sec
TDH	Total dynamic head	ft
V_t	Total cycling storage volume	ft^3
V_x	Individual pump cycling volumes	ft^3
W	Minimum required distance between pumps	ft
Y	Minimum level floor distance upstream of pump	ft

1 Ft^3 = 7.481 gallons
1 cfs = 448 gpm

14.3 Design Considerations

14.3.1 Location

Economic and design considerations dictate that the pump station be located relatively near the low point of the highway. Hopefully a frontage road or overpass is available for easy access to the station. The station and access road should be located on high ground so that access can be obtained if the highway becomes flooded. Soil borings should be made during the selection of the site to determine the allowable bearing capacity of the soil and to identify any potential problems

Decisions regarding location and building architecture need to be made during the concept phase so station design can proceed in an efficient manner. Following are some items that are to be considered for locating the pump station site:

- Building and site access is safe and monitorable.
- Pump stations shall be situated so that the facility is accessible during storm events up to the 100-year event.
- Ample parking and working areas should be provided adjacent to the station for maintenance and repair vehicles.
- The station facade and grounds are aesthetically compatible with the surrounding area.

14.3.2 Design Capacity

Frequency

The design capacity of a pump station is determined by its location within the drainage system. It is ADOT practice that pump stations and appurtenant storage system for draining roadway sumps that are considered "depressed" shall accommodate the inflow for a 50-year storm event in a manner that meets the design spread criteria. It is desirable to evaluate the total drainage system for the 100-year storm event to determine the extent of flooding and the associated risk.

Contributing Area

Hydrologic design should be based on the ultimate development of the area that must drain to the station. Every consideration should be made to keep the contributing drainage area as small as possible. Water that originates outside of the depressed areas should not be allowed to enter the depressed areas because of the need to pump all of this water. Only flows that cannot be by-passed or passed through should be collected. The contributing drainage area should be isolated to prevent off-site flows being diverted to the pump station.

Storage

Storage, in addition to that which exists in the wet well, should be evaluated at all pump station sites. For most highway pump stations, the high flows of the inflow hydrograph will occur over a relatively short time. Additional storage will greatly reduce the peak pumping rate required. An economic analysis can be used to determine the optimum combination of storage and pumping capacity. Since most highway-related pump stations are associated with a localized low point, it is not reasonable to consider above ground storage. The simplest form of storage for these depressed situations is either the enlargement of the collection system or the construction of an underground storage facility or a combination of the two. These can typically be constructed under the roadway area and will not require additional right-of-way.

14.3 Design Considerations (continued)

14.3.2 Design Capacity (continued)

When storage is used to reduce peak flow rates, a routing procedure must be used to design the system. To determine the discharge rate, the routing procedure integrates three independent elements: the inflow hydrograph, the stage-storage relationship, and the stage-discharge relationship.

14.3.3 Pit Types

Basically, there are two types of stations: wet-pit and dry-pit.

Wet-Pit Stations - In the wet-pit station, the pumps are submerged in a wet well or sump with the motors and the controls located overhead. With this design, the stormwater is pumped vertically through a riser pipe. The motor is commonly connected to the pump by a long drive shaft located in the center of the riser pipe. See Figure 14-2 for typical layout.

Dry-Pit Stations - Dry-pit stations consist of two separate elements: the storage box or wet well and the dry well. Stormwater is stored in the wet well which is connected to the dry well by horizontal suction piping. Centrifugal pumps are usually used. Power is provided by either close-coupled motors in the dry well or long drive shafts with the motors located overhead. The main advantage of the dry-pit station for stormwater is the availability of a dry area for personnel to perform routine and emergency pump and pipe maintenance. See Figure 14-3 for typical layout.

Roof Hatches And Monorails

It will be necessary to remove motors and pumps from the station for periodic maintenance and repair. Removable roof hatches located over the equipment is a cost-effective way of providing this capability. Mobile cranes can simply lift the equipment directly from the station onto maintenance trucks. Monorails are usually more cost-effective for larger stations.

Wet Pit Size

In the wet pit, the plan area must accommodate the distance between the pumps, the clearance to the sidewalls, the flow length approaching the pump, and the minimum distance from floor to the underside of the bell. The station depth should be minimum. No more depth than that required for pump submergence and clearance below the inlet invert is necessary, unless foundation conditions dictate otherwise. For the design event, typically the highway operational frequency, the top water surface shall be no higher than that which will result in a hydraulic grade line at the critical inlets with a freeboard of 0.5 foot to the gutter elevation.

14.3 Design Considerations (continued)

14.3.3 Pit Types (continued)

Method of Construction
The method of construction has a major impact on the construction cost of pump station. For stormwater pump station, yearly operating costs are usually insignificant compared to construction costs. Therefore, the type of construction should be chosen carefully, between open-pit construction or braced construction. Soil conditions are the primary factor in selecting the most cost-effective alternative.

Recommendation for Design
Since dry-pit stations are as much as 60% more expensive than wet-pit stations, wet-pit stations are recommended.

14.3.3.1 ADOT Freeway Design Practice

Use a wet well design. Wet well sizing is based on guidelines from the **Hydraulic Institute Standards**. The wet-well must be sealed from the engine room and control room. The wet well area will include explosion proof lighting. A gas tight seal shall be provided between the wet well and the engine area. Access to the wet well will be by a separate enclosed stairway that has a push/pull intake and exhaust system.

14.3.4 Pump Types

14.3.4.1 Flow Types

The most common types of stormwater pumps are axial flow (propeller), radial flow (impeller) and mixed flow (combination of the previous two). Each type of pump has its particular merits. It is difficult to have a totally objective selection procedure. Cost, reliability, operating and maintenance requirements are all important considerations when making the selection. First costs are usually of more concern than operating costs in stormwater pump stations since the operating periods during the year are relatively short. Ordinarily, first costs are minimized by providing as much storage as possible, with two or three small pumps.

Axial Flow Pumps - Axial flow pumps lift the water up a vertical riser pipe; flow is parallel to the pump axis and drive shaft. They are commonly used for low head, high discharge applications. Axial flow pumps do not handle debris particularly well because the propellers will bend or possibly break if they strike a relatively large, hard object. Also, fibrous material will wrap itself around the propellers.

Radial Flow Pumps - Radial flow pumps utilize centrifugal force to move water up the riser pipe. They will handle any range of head and discharge, but are the best choice for high head applications. Radial flow pumps generally handle debris quite well. A single vane, non-clog impeller handles debris the best because it provides the largest impeller opening. The debris handling capability decreases with an increase in the number of vanes since the size of the openings decrease.

14.3 Design Considerations (continued)

14.3.3.1 Pit Type (continued)

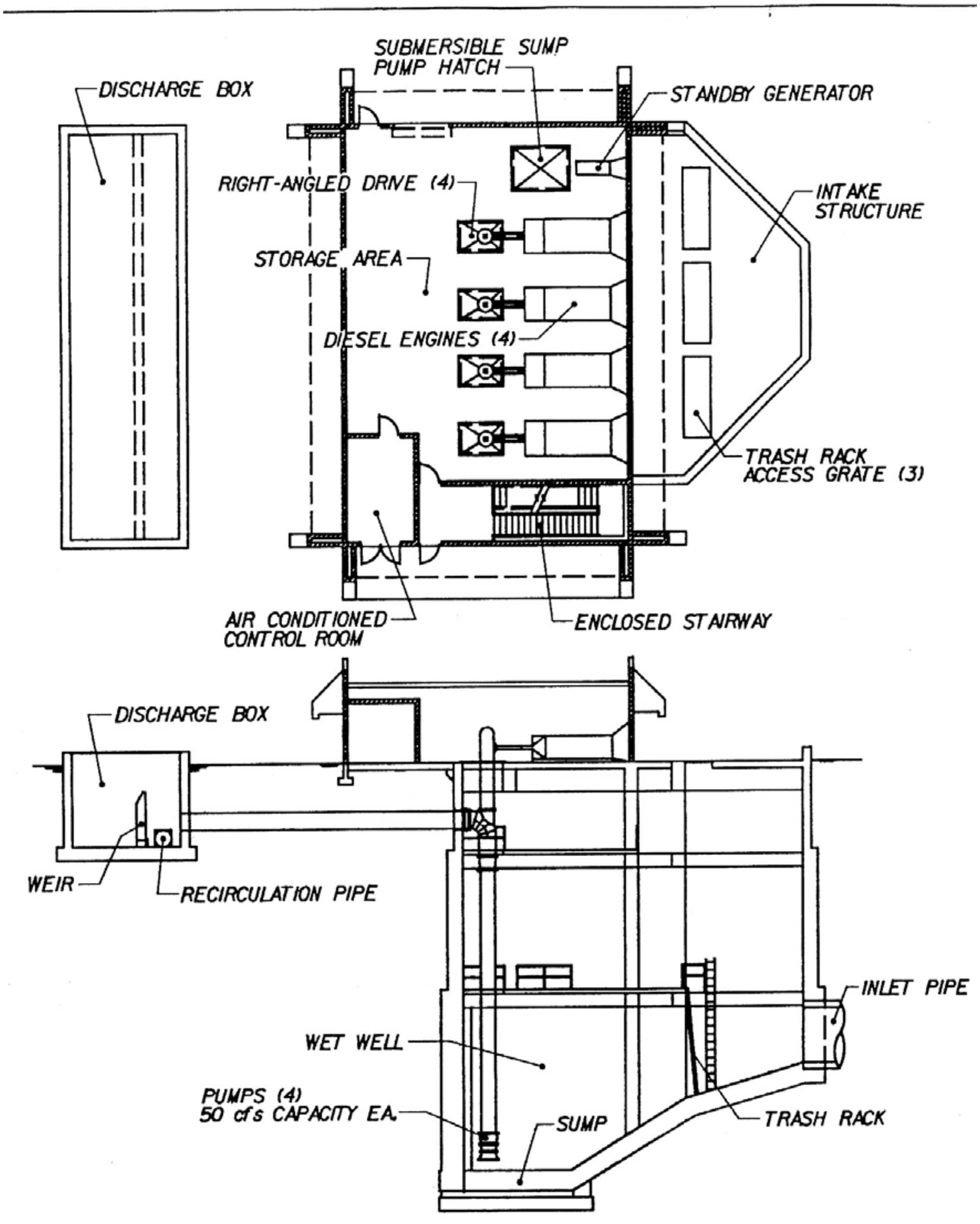


Figure 14-2 Typical Wet-Pit Station

14.3 Design Considerations (continued)

14.3.3.1 Pit Type (continued)

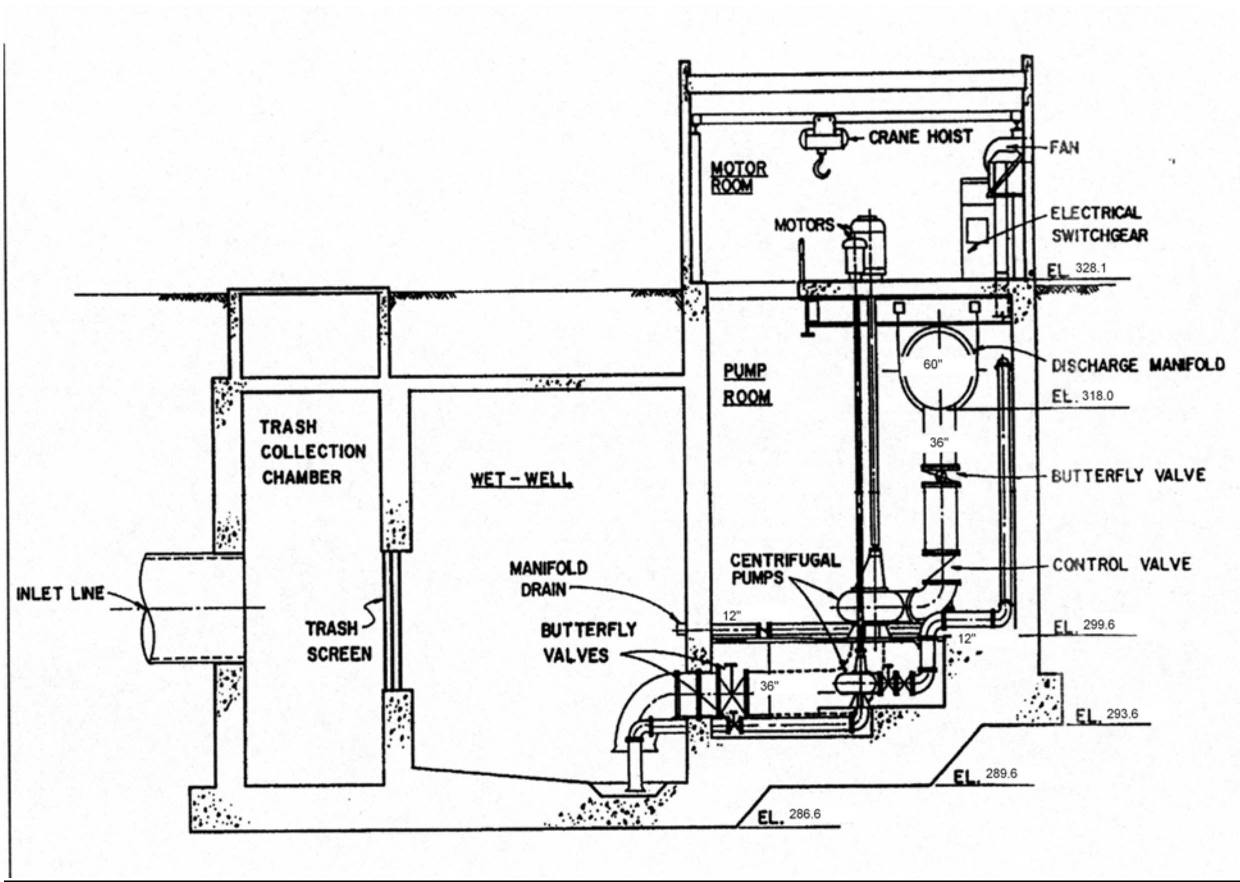


Figure 14-3 Typical Dry-Pit Station

Source: FHWA IP-82-17

14.3 Design Considerations (continued)

14.3.4.1 Flow Types (continued)

Mixed Flow Pumps - Mixed flow pumps are very similar to axial flow except they create head by a combination of lift and centrifugal action. An obvious physical difference is the presence of the impeller "bowl" just above the pump inlet. They are used for intermediate head and discharge applications and handle debris slightly better than propellers. These pumps can be driven by motors or engines housed overhead or in a dry well or by submersible motors located in a wet well. Submersible pumps frequently provide special advantages in simplifying the design, construction, maintenance and, therefore, cost of the pumping station. Use of anything other than a constant speed, single stage, single suction pump would be rare. Pumps shall be capable of handling solids 3” or smaller.

14.3.4.2 Submergence

Submergence is the depth of water above the pump inlet necessary to prevent cavitation and vortexing. It varies significantly with pump type, speed, inlet bell diameter and atmospheric pressure. This dimension is provided by the pump manufacturer and is determined by laboratory testing. A very important part of submergence is the required net positive suction head (NPSH) because it governs cavitation. The available NPSH should be calculated and compared to the manufacturer's requirement. Additional submergence may be required at higher elevations. As a general rule, radial flow pumps require the least submergence while axial flow pumps require the most.

One popular method of reducing the submergence requirement (and therefore the station depth) for axial and mixed flow pumps, when cavitation is not a concern, is to attach a suction umbrella. A suction umbrella is a dish-shaped steel plate attached to the pump inlet that improves the entrance conditions by reducing the intake velocities. For umbrella velocities of 2 ft/sec or less, a submergence to pump bell diameter ratio of 0.80 can be used. Required submergence criteria developed by the Hydraulic Institute is shown in Figure 14-4 as presented in “American National Standard for Pump Intake Design”.

14.3 Design Considerations (continued)

14.3.4.2 Submergence (continued)

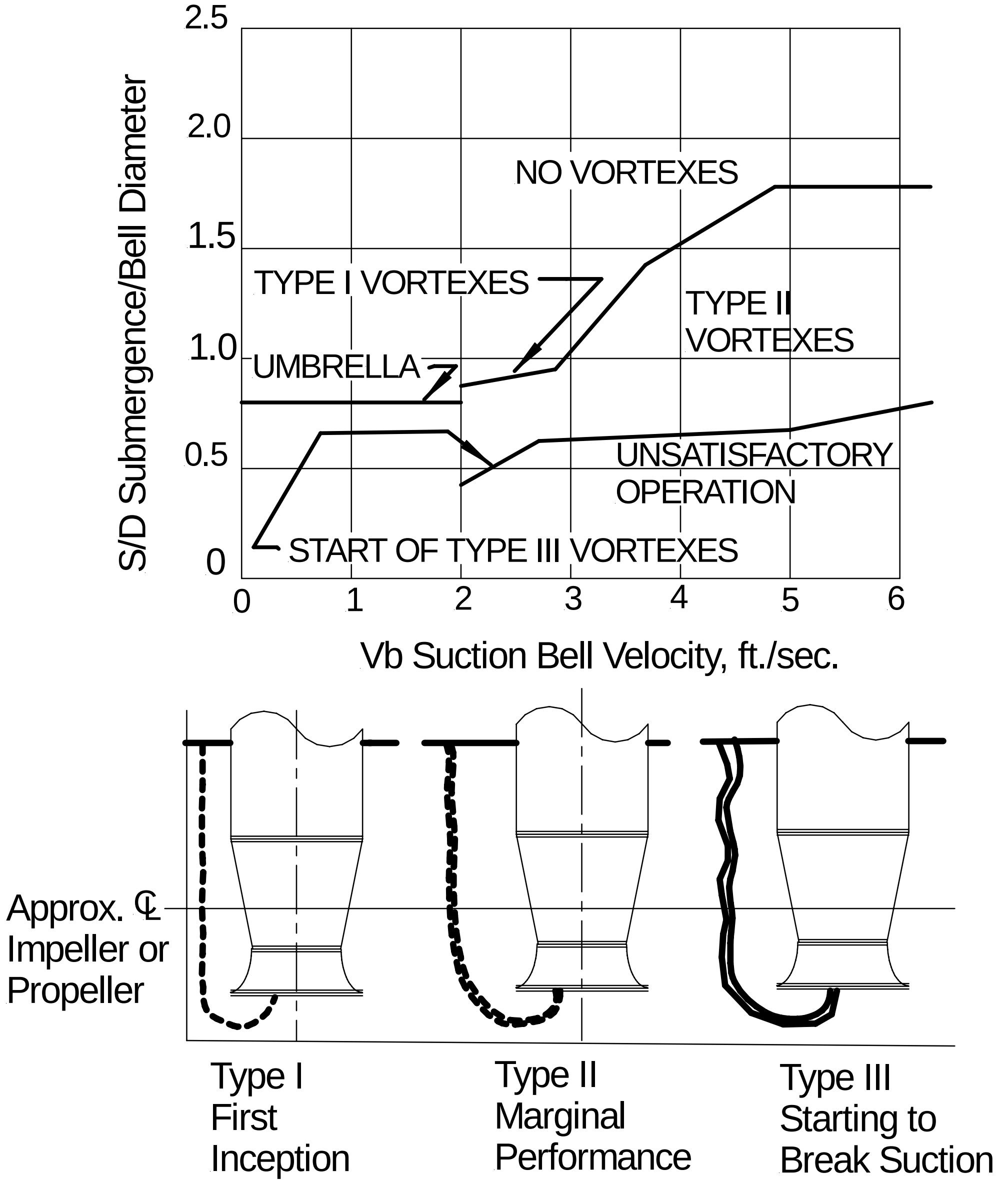


Figure 14-4 Vortex Description And Submergence Requirements

14.3 Design Considerations (continued)

14.3.4.3 Standby/Spare Pumps

Considering the short duration of high inflows, the relative infrequency of the design storm event, the odds of a malfunction and the typical consequences of a malfunction, spare or standby pumps are not warranted in stormwater applications. If the consequences of a malfunction are particularly critical, it is more appropriate to add another main pump and reduce their size accordingly.

14.3.4.4 Sump Pumps

It is ADOT preference for freeway installations to use a station electric “trash” sump pump that has approximately one-half the capacity of the primary pumps. The pump shall be capable of handling four-inch solids.

14.3.4.5 ADOT Freeway Design Practice

Use a long shaft vertical turbine. The pump assembly consists of a bowl assembly with impeller, a long shaft, and a pipe column with an attached elbow. The long shaft will require lubrication at each bearing. This is to be provided by an automatic system that provides lubricant at each start.

14.3.5 Power

14.3.5.1 General

Several types of power may be available for a pump station. Examples are electric motors, and diesel or natural gas engines. The maintenance engineer should be contacted for input in the selection process. The designer should select the type of power that best meets the needs of the project based on an estimate of future energy considerations and overall station reliability. A comparative cost analysis of alternatives is helpful in making this decision. The need for backup power is dependent upon the consequences of failure. The decision to provide it should be based on economics and safety.

Electric Motors - Back-up/Standby Power

For electric motors, two independent electrical feeds from the electric utility with an automatic transfer switch may be the cost-effective choice when backup power is required. A standby generator is generally less cost-effective because of its initial costs. Also, standby generators require considerable maintenance and testing to ensure operation in times of need.

For extensive depressed freeway systems involving a number of electric motor-driven stations, a mobile generator may be the cost-effective choice for backup power. A trailer-mounted generator can be stored at any one of the pump stations. If a power outage occurs, maintenance forces can move the generator to the affected station to provide temporary power. If a mobile generator is used as the source of backup power, it may be necessary to add additional storage to compensate for the time lag that results in moving the generator from site to site. This lag can typically be 1.0 to 1.5 hours from the time the maintenance forces are notified.

14.3 Design Considerations (continued)

14.3.5.1 General (continued)

Consideration should be given to whether the pump station is to have standby power (SBP). If it is preferred that stations have a SBP receptacle, manual transfer switch and a portable engine/generator set, then the practical power limit of the pumps becomes 55 kW (75 HP) since this is the limit of the power generating capabilities of most portable generator units.

14.3.5.2 ADOT Freeway Design Practice

Diesel or natural gas engines driving an electric generator are preferred. The standard engine is non-turbocharged, water-cooled, operating at 1200 rpm. Engine exhaust must be equipped with mufflers and be directed in a way to reduce noise impacts to residences and/or businesses. A minimum clearance of 4 feet around all pumps, engines, and other mechanical equipment shall be provided in the station design.

14.3.6 Monitoring and Control

14.3.6.1 General

Pump stations are vulnerable to a wide range of operational problems from malfunction of the equipment to loss of power. Monitoring systems such as on-site warning lights and remote alarms can help minimize such failures and their consequences.

Telemetering is an option that should be considered for monitoring critical pump stations. Operating functions may be telemetered from the station to a central control unit. This allows the central control unit to initiate corrective actions immediately if a malfunction occurs. Such functions as power, pump operations, unauthorized entry, explosive fumes and high water levels can be monitored effectively in this manner. Perhaps the best overall procedure to assure the proper functioning of a pump station is the implementation of a regular schedule of maintenance conducted by trained, experienced personnel.

Instruments such as hour meters and number-of-starts meters should be used on each pump to help schedule maintenance. Input from maintenance forces should be a continuous process so that each new generation of stations will be an improvement.

Water-Level Sensors

The water-level sensors activate the pumps and, therefore, are a vital component of the control system. There are a number of different types of sensors that can be used. Types include the float switch, electronic probes, ultrasonic devices, mercury switch and air pressure switch. The location or setting of these sensors control the starting and stopping of the pump motors. Their function is critical because the pump motors or engines must not start more frequently than an allowable number of times per hour (i.e., the minimum cycle time) to avoid damage. To prolong the life of the motors, sufficient volume must be provided between the pump start and stop elevations to meet the minimum cycle time requirement. The on-off setting for the first pump is particularly important because it defines the most frequently used cycle.

14.3 Design Considerations (continued)

14.3.6.2 ADOT Freeway Design Practice

Control System
A control room separate from the wet well or engine/pump room shall contain the control equipment. The control room is completely isolated from both the wet well and the engine room. It is separately air-conditioned. The room shall be temperature controlled. Should the station electric power fail, the stand-by generator will be activated, and the generator will continue to drive the control room air-conditioner, as a well as the other ventilation systems.

The sump pump will always be the first pump on and the last pump off. The four primary pumps will be consecutively lagged as the water level rises in the wet well. The order of lagging shall be by manual selection.

Uninterruptible Power Supply (UPS)
UPS shall be provided to furnish power for controlling and monitoring the pump station. Batteries sized for one-half hour outage at full capacity will supply power to the UPS. A signal from the UPS is provided to start a stand-by engine generator set when there is a power outage. An adjustable time will delay start of the standby generator. The UPS will be equipped with a static transfer switch, a manual maintenance switch, metering, and alarms and status lights.

Emergency/Stand-by Power Supply
A diesel engine-generator set is provided to provide all electrical station power and air-conditioning in the event of commercial power outage. The stand-by generator also is used to recharge the UPS system. The stand-by engine/generator set will start on a signal from the UPS after a set delay. Power is provided to the UPS via an automatic transfer switch.

Sensors and Monitors
For each engine driven pump and the sump pump a level-sensing probe located in the wet well will provide signals to start and stop. Pilot lights on a control panel will indicate the status of each pump. Any pump running will also actuate external signals to signify the station is operating. Sensors are attached at each of the discharge line flap gates to determine if the pump station is discharging. A sensor shall be provided in the discharge box to measure the head at the weir. The head shall be converted to discharge. Means will be provided to transmit the pump on/off and discharge status to a central location.

Probes will be provided for low water and high water conditions. Either of these conditions will activate local and remote indicators. These two alarms will only be capable of being reset by manual means at the pump station control panel.

14.3 Design Considerations (continued)

14.3.6.2 ADOT Freeway Design Practice (continued)

Communications Link

A reliable means of communicating the status of the following conditions is to be provided:

1.) Pump station discharge
2.) High level in wet well
3.) Pump failure
4.) Pump status (on/off)
5.) Sump pump (overload, on/off)
6.) Engine failure
7.) Engine status (on/off)
8.) Fuel level in tanks (level and low-level alarm)
9.) Control room power failure
10.) Utility power failure
11.) Uninterruptible Power Supply (failure, low battery)
12.) Stand-by generator (malfunction, on/off)
13.) Engine room and wet well high temperature alarm
14.) Fans in wet well, stairway, engine room (overload, on/off)
15.) Combustible gas (wet well and/or engine room)
16.) Fire
17.) Low foam liquid level (indicates fire suppression system activated)
18.) Security violated

14.3 Design Considerations (continued)

14.3.7 Collection System

14.3.7.1 General

Storm drains leading to the pumping station are usually designed on a flat grade to minimize depth and cost. A minimum grade that produces a velocity of 3 ft/sec in the pipe while flowing at a depth of one-quarter (1/4) full is suggested to avoid siltation problems in the collection system. Minimum cover or local head requirements will usually govern the depth of the uppermost inlets. Storm drainage systems tributary to the pump station can be quite extensive and costly. Linear or intermediate storage along the storm drain may be used to reduce peak flows and pipe sizes. For some pump stations, the storage available in the collection system may be significant. However, it is often necessary to provide additional storage near the pump station. This may be done by oversizing the collection system or designing an underground vault.

The collector lines should preferably terminate at a forebay or storage box. However, they may discharge directly into the station. Under the latter condition, the capacity of the collectors and the storage within them is critical to providing adequate cycling time for the pumps and must be carefully calculated. The inlet pipe should enter the station perpendicular to the line of pumps. It should be aligned with the centerline of the wet well and should have a straight run of at least 100 feet before entering the station. The inflow should distribute itself equally to all pumps. Baffles may be required to ensure that this is achieved.

Grate inlets and/or slotted drain inlets are recommended for lines that connect to pump stations. They will act as screens to prevent large objects from entering the system and possibly damaging the pumps. This approach has an additional advantage of simplifying debris removal since debris can be more easily removed from the roadway than the wet well.

14.3.7.2 Trash Racks And Grit Chambers

Trash racks should be provided at the entrance to the wet well if large debris is anticipated. For stormwater pumping stations, simple steel bar screens are adequate. Usually, the bar screens are inclined with bar spacing approximately 1.5 in. Constructing the screens in modules facilitate removal for maintenance. An emergency overflow should be provided to protect against clogging and subsequent surcharging of the collection system.

14.3 Design Considerations (continued)

14.3.8 Discharge System

14.3.8.1 General

The discharge piping should be kept as simple as possible. Pumping systems that lift the stormwater vertically and discharge it through individual lines to a gravity storm drain as quickly as possible are preferred. The pump location with respect to the outfall chamber should be set to provide as short a distance as possible. Individual pump discharge lines are the most cost-effective system for short outfall lengths. Individual lines may exit the pumping station either above or below grade. Damaging pump reversal could occur with very long force mains. Check valves should be installed. The effect of stormwater returning to the sump after pumping stops should be considered.

Gate valves should be provided in each pump discharge line to provide for continued operation during periods of repair, etc. A cost analysis should be performed to determine what length and type of discharge piping justifies a manifold. Number of valves required shall be kept to a minimum to reduce cost, maintenance and headloss through the system.

It may be necessary to pump to a higher elevation using long discharge lines. This may dictate that the individual lines be combined into a force main via a manifold. For such cases, check valves must be provided on the individual lines to keep stormwater from running back into the wet well and restarting the pumps or prolonging their operation time. Check valves should preferably be located on horizontal layouts rather than vertical, to prevent sedimentation on the downstream side after the valve closing.

The discharge line is typically either steel or cast iron and must be sized to be at least the size of the pump discharge diameter. The need for a larger pipe required to limit the maximum velocity to 10 ft/sec should be checked by the following equation.

$$D= 1.128(Q/V)^{0.5}$$

14.3.8.2 Flap Gates And Valving

Flap Gates - The purpose of a flap gate is to restrict water from flowing back into the discharge pipe and to discourage entry into the outfall line. Flap gates are usually not watertight so the elevation of the discharge pipe should be set above the normal water levels in the receiving channel. If flap gates are used, it may not be necessary to provide for check valves.

Check Values - Check valves are watertight and are required to prevent backflow on force mains that contain sufficient water to restart the pumps. They also effectively stop backflow from reversing the direction of pump and motor rotation. They must be used on manifolds to prevent return flow from perpetuating pump operation. Check valves should be "non-slam" to prevent water hammer. Types include: swing, ball, dashpot and electric.

Gate Valves – A gate valve is used as a shut-off device on force mains to allow for pump or valve removal. These valves should not be used to throttle flow. They should be either totally open or totally closed.

14.3 Design Considerations (continued)

14.3.8.2 Flap Gates And Valving (continued)

Air/Vacuum Valves - Air/Vacuum valves are used to allow air to escape the discharge piping when pumping begins and to prevent vacuum damage to the discharge piping when pumping stops. They are especially important with large diameter pipe. If the pump discharge is open to the atmosphere, an air-vacuum release valve is not necessary. Combination air release valves are used at high points in force mains to evacuate trapped air.

14.3.8.3 ADOT Freeway Design Practice

The pump discharge line is connected to a discharge box. The box drains to the discharge outfall or storm drain. The discharge box is divided by a low wall that performs as a weir. The wall also creates a chamber that provides storage for pump recirculation operation. The chamber for re-circulation is sized for testing and exercising one pump/engine set or the sump pump. The wall is beveled to form a sharp crested weir.

Two manually operated sluice gates are provided; one between the recirculation chamber and the outlet chamber and one at the recirculation pipe to prevent backflow during normal operations. A flap gate, usually of cast iron, is provided for each pump discharge line.

14.3.9 Safety

14.3.9.1 General

All elements of the pump station should be carefully reviewed for safety of operation and maintenance. Ladders, stairwells and other access points should facilitate use by maintenance personnel. Adequate space should be provided for the operation and maintenance of all equipment. Particular attention should be given to guarding moving components such as drive shafts and providing proper and reliable lighting. It may also be prudent to provide air-testing equipment in the station so maintenance personnel can be assured of clean air before entering.

Pump stations may be classified as a confined space in which case access requirements along with any safety equipment are all defined by code. Pump stations should be designed to be secure from entry by unauthorized personnel and as few windows as possible should be provided.

The designer should be aware of established safety procedures for working in pump stations. The designs shall meet the established safety criteria and enhance the ease of compliance

14.3.9.2 Internal Environment

Ventilation

Ventilation of dry and wet wells is necessary to ensure a safe working environment for maintenance personnel. The ventilation system can be activated by a combined light/ventilation switch at the entrance to the station. Maintenance procedures normally require personnel to wait several minutes after ventilation has started before entering the well. The testing of the air in the wet well prior to allowing entry may be

14.3 Design Considerations (continued)

14.3.9.2 Internal Environment (continued)

required. If mechanical ventilation is required to prevent buildup of potentially explosive gasses, the pump motors or any spark producing equipment should be rated explosion proof or the fans run continuously.

Heating and dehumidifying requirements are variable. Their use is primarily dependent upon equipment and station type, environmental conditions and station use.

14.3.9.3 Hazardous Spills

The possibility of hazardous spills is always present under highway conditions. In particular, this has reference to gasoline, and the vulnerability of pump stations and pumping equipment to fire damage. There is a history of such incidents having occurred and also of spills of oils, corrosive chemicals, pesticides and the like having been flushed into stations, with undesirable results. The usual design practice has been to provide a closed conduit system leading directly from the highway to the pump station without any open forebay to intercept hazardous fluids, or vent off volatile gases. With a closed system, there must be a gas-tight seal between the pump pit and the motor room in the pump station. Preferably, the pump station should be isolated from the main collection system and the effect of hazardous spills by a properly designed storage facility upstream of the station. This may be an open forebay or a closed box below the highway pavement or adjacent to it. The closed box must be ventilated by sufficient grating area at each end.

14.3.9.4 ADOT Freeway Design Practice

Combustible Gas Detection
Combustible gas detection systems shall be provided for the wet well and engine room. The systems shall include self-contained gas monitors and remote detectors. The systems shall be powered by the UPS. The systems shall be capable of responding to at least two levels of the presence of combustible gases. The lower level shall send signals to the communication system, actuate local signals, and activate the ventilation system. The high level sensor shall shut-off any engines that are running and activate the foam fire suppression system.

Fire Suppression System
The foam fire suppression system shall be capable of three 10-minute applications.

Ventilation System
The station exhaust fan ventilation system is composed of three separate areas. One each is provided for the wet well, engine room and stairway. The wet well system has a push/pull intake and exhaust system shall provide for 12 air changes per hour. As noted above, this system is activated by either manually or by gas detectors in the wet well.

The engine room has two exhaust fans that are thermostatically controlled. They are set to turn on when the room temperature exceeds 90 and 95 degrees Fahrenheit and the room temperature is higher than the outside ambient temperature. As noted above, this system is activated by either manually or by gas detectors in the wet well or engine room.

14.3 Design Considerations (continued)

14.3.9.4 ADOT Freeway Design Practice (continued)

The stairway has push/pull intake and exhaust systems that exhaust air from four levels. The stairwell light switch activates the stairwell ventilation system. The stairwell is equipped with fire doors at the top and bottom entrances. Each exhaust duct is equipped with a fire damper that closes automatically when smoke is detected in the wet well or engine room.

14.3.10 Building Architecture

14.3.10.1 General

The appearance of the pump station should be compatible with the surrounding area. The site layout should provide a secure, easily monitored structure. Applicable building codes shall be followed. The internal areas of the pump station are divided into the wet well, the engine/pump room, the control room, and an enclosed stairwell that provides access to the wet well. Items of particular concern regarding building architecture are the fire protection, security, and air-conditioning and ventilation systems.

14.3.10.2 Building Access

Separate access to the building will be provided for personnel into the engine/pump room, the control room, and the stairwell. In addition to the personnel access doors, access for pick-up or other light to medium sized vehicles shall be provided by an 8 ft by 8 ft. roll up door into the engine/pump room and through double door into the control room. A roof access hatch over each engine/pump shall be provided.

14.3.11 Site Design

Site location and layout is important to achieve a station that is secure and compatible with the surrounding area.

14.3.11.1 Access

The site should be accessed from a frontage road or a local street. It should not require access from a limited access highway.

14.3.11.2 Security

The area is to be well lighted. An eight-foot high wall shall surround the station with access only through a 24-foot locked gate. The access gate shall be situated so as to allow maximum visibility of as many accessible entrances as possible. If this is not possible, provide an open-weave fence section in the wall to allow observation of doors not visible from the access gate. All accessible exterior doors shall be equipped with proximity sensors that will activate local and remote alarms.

14.3 Design Considerations (continued)

14.3.11.3 Fuel Storage

Generally the fuel storage tanks are located along the block wall. The location must be accessible for refueling and maintenance, but out of the way for other operations. Diesel fuel storage tanks are to have 24-hour capacity. Storage tanks may be in-ground tanks located in a vault or above ground steel-lined concrete. Vaults are to be provided with a roof deck to block solar radiation and water intrusion. The vault will provide for containment of 110 percent of the volume of the diesel tanks. The vault will not have a drain; any accumulation of liquid must be pumped out with a portable pump. Above ground concrete tanks must be fire rated and ballistic proof.

14.3.11.4 Utilities

Required utilities are electricity and water. Sanitary sewers are not required. The minimum electric service is 480 volt, 3-phase, four wire. The minimum size of water main is 6-inch diameter. A fire hydrant should be provided near the pump station and a minimum 4-inch service connection with back- flow preventor should be provided to the pump station building.

14.3.11.5 Drainage

Adequate provisions for site drainage shall be provided. Local drainage can be connected to the discharge box or outlet pipe. On-site drainage should NOT be allowed to enter the pump station and require pumping.

14.4 Design Concepts and Criteria

The following discussion is made with the objective of minimizing the construction, operation and maintenance costs of highway stormwater pump stations while remaining consistent with the practical limitations of all aspects.

14.4.1 Discharge Head And System Curve

14.4.1.1 Headloss

Stormwater pumps are extremely sensitive to changes in head requiring the head demand on the pumps be calculated as accurately as possible. All valve and bend losses are to be considered in the computations. In selecting the size of discharge piping, consideration should be given to the manufactured pump outlet size vs. the head loss produced by smaller piping. This approach should identify a reasonable compromise in balancing cost. Once the head losses have been calculated for the range of discharges expected, the system curve (Q vs. TDH) can be plotted. This curve defines the energy required to pump any flow through the discharge system. It is especially critical for the analysis of a discharge system with a forcemain. When overlaid with pump performance curves (provided by manufacturer), it will yield the pump operating points (see Figure 14-5).

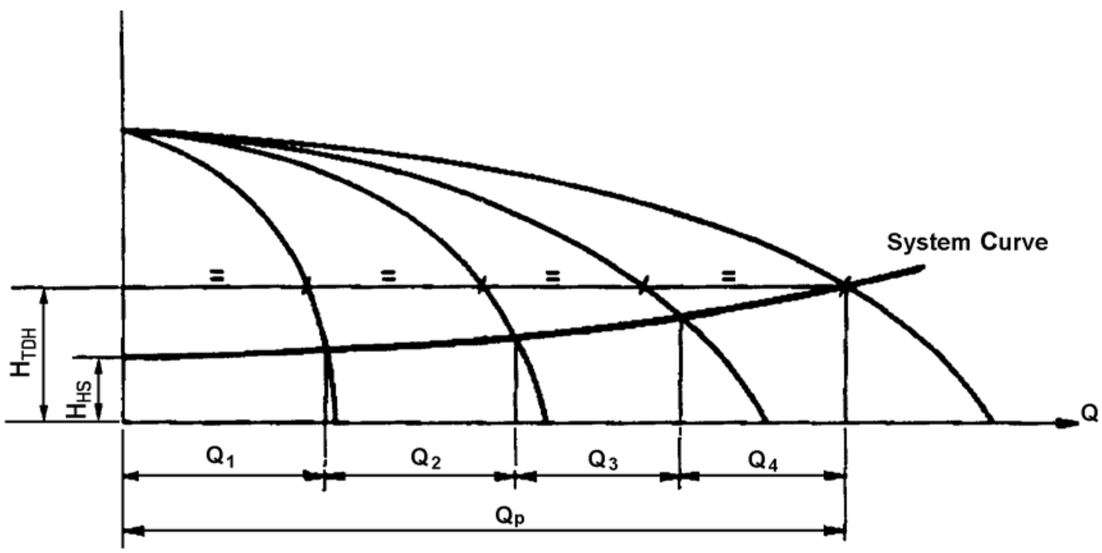


Figure 14-5 Systems Curve

The combination of static head, velocity head and various head losses in the discharge system due to friction is called total dynamic head. These head losses are minimized by the selection of correctly sized discharge lines and other components.

Losses in pipe system:	K
Elbow, 90 degree	0.20
Miscellaneous couplings and fittings	0.05
Flap Gate	0.25
Outlet	1.00

14.4 Design Concepts and Criteria (continued)

14.4.1.2 Total Dynamic Head

Pumps for a given station are selected to all operate together to deliver the Design Q at a Total Dynamic Head computed to correspond with the Design Water Level. Because pumps must operate over a range of water levels, the quantity delivered will vary significantly between the low level of the range and the high level.

A curve of total dynamic head versus pump capacity is available for each pump from the manufacturer. When running, the pump will respond to the total dynamic head prevailing and the quantity of discharge will be in accordance with the curve. The designer must study the pump performance curves for various pumps in order to develop an understanding of the pumping conditions (head, discharge, efficiency, kilowatt, etc.) throughout the full range of head that the pump will operate under. The system specified must operate properly under the full range of specified head.

When the pump is raising the water from the lowest level, the static head will be greatest and the discharge will be the least. When operating at the highest level, the static head will be the least and the discharge will be the greatest. Pump capabilities must always be expressed in both quantity of discharge and the total dynamic head at a given level. Typically, these conditions are specified for three points on the performance curve. One point will be near maximum TDH, the next will be the design point, and the third will be at about minimum TDH.

A pump is selected to operate with the best possible efficiency as its Design Point, corresponding to the Design Water Level of the station, and its performance is expressed as the required discharge at the resulting total dynamic head. The efficiency of a stormwater pump at its design point may be 75 or 80% or more, but this will depend on the type of pump. When the static lift is greatest (low water in sump), the energy required (horsepower) maybe the greatest even though the quantity of water raised is less. This is because the pump efficiency may also be much less. The pump selection should be made so that maximum efficiency is at the design point.

The total dynamic head (TDH) should be determined for a sufficient number of points to draw the system head curve that will be discussed later in this chapter. The TDH is computed as follows:

$$\mathbf{TDH = H_s + H_f + H_v + H_l} \qquad \mathbf{(14.1)}$$

where: TDH = total dynamic head, ft
H_s = max. static head (at lowest pump-off elevation), ft
H_f = friction head, ft (i.e., friction loss)
H_v = velocity head, ft ($V^2/2g$)
H_l = losses through fittings, valves, etc., ft

Adjustments may have to be made to these curves to account for losses within the pumping unit provided by the manufacturer.

14.4 Design Concepts and Criteria (continued)

14.4.2 Pumps

14.4.2.1 Main Pumps

Number And Capacity

The number of pumps needed are determined by following a systematic process defined in section 14.6. However, two to three pumps have been judged to be the recommended minimum. If the total discharge to be pumped is small and the area draining to the station has little chance of increasing substantially, the use of two pumps in one station is preferred. Consideration may be given to over sizing the pumps to compensate, in part, for a pump failure. A two-pump system could have pumps designed to pump 66 - 100% of the required discharge and the three pump system could be designed so that each pump will pump 50% of the design flow. The possible impacts caused by the loss of one pump can be used as a basis for deciding the size and numbers of the pumps.

Economic limitations on power unit size as well as practical limitations governing operation and maintenance should be used to determine the upper limit of pump size. The minimum number of pumps used may increase due to these limitations.

Equal-size pumps are to be used. Identical size and type enables all pumps to be freely alternated into service. This equalizes wear and reduces needed cycling storage. It also simplifies scheduling maintenance and allows pump parts to be interchangeable. Hour meters and start meters are to be provided, these aid in scheduling needed maintenance.

Final Selection

For the typical highway application, any of the three pump types described earlier will usually suffice. If not, manufacturers' information will likely dictate the type required. However, knowing the operating RPMs, a computation for specific speed can be made to check the appropriateness of the pump type (see Figure 14-6 to determine the ranges where specific impeller types should be used). Suction specific speed may be defined as that speed in revolutions per minute at which a given impeller would operate if reduced proportionally in size so as to deliver a capacity of 3.3 gpm against TDH of 3 ft. This is an index number descriptive of the suction characteristics of a given pump. Higher numerical values are associated with better NPSH capabilities. This number should be checked for dry pit applications and systems with suction lifts. Once the pump type and capacity have been determined, the final selection of the pump can be made.

14.4 Design Concepts and Criteria (continued)

14.4.2.1 Main Pumps (continued)

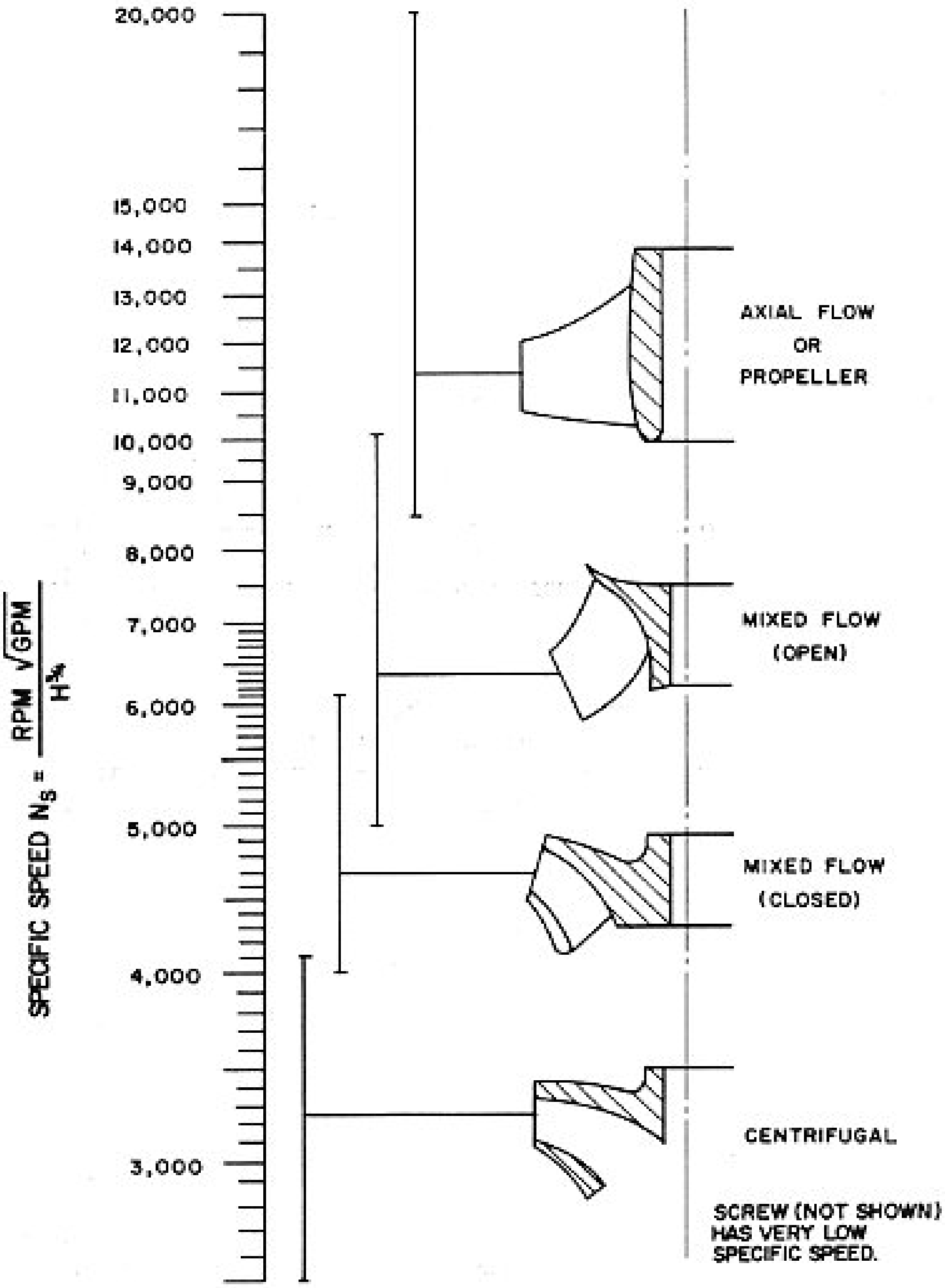


Figure 14-6 Specific Pump Speed vs. Impeller Types

14.4 Design Concepts and Criteria (continued)

14.4.3 Wet Well Design

The primary criteria for sizing the wet well involves the number of pumps and pump bell diameter. This criteria includes floor clearance, minimum distance between an inlet bell and the wall, the minimum clearance between adjacent inlet bells, width of partition wall between pumps, and the submergence required for the pump bell diameter. The specific criteria for both circular and rectangular wet well can be found in “American National Standard for Pump Station Intake Design” by the Hydraulic Institute as well as from pump manufacturers.

The wet well size and shape are important factors for both their contribution to available storage and for providing room fro proper sizing and layout of pumps. However, the final number of pumps is not normally known until the final design phase. Therefore, it is necessary to estimate wet well dimensions based on a trial number and size of pumps. It may be necessary to increase dimensions to provide additional storage or to accommodate additional pumps.

14.4.3.1 Cycling Sequence And Volumes

Cycling is the starting and stopping of pumps, the frequency of which must be limited to prevent damage and possible malfunction. Sufficient volume must be provided either in the wet well or outside the wet well for safe cycling. The volume required to satisfy the minimum cycle time is dependent upon the characteristics of the power unit, the number and capacity of pumps, the sequential order in which the pumps operate and whether or not the pumps are alternated during operation. However, to keep sediment in suspension, the wet well volume should not be oversized

Before discussing pump cycling considerations, operation of a pump station will be described. Initially, the water level in the storage basin will rise at a rate depending on the rate of the inflow and physical geometry of the storage basin. When the water level reaches the stage designated as the first pump start elevation, the pump will be activated and discharge water from storage at its designated pumping rate. For pumps that are engine driven, there will be a delay between the initial start signal and the commencement of pumping. This delay may be on the order of 90 seconds. The volume of inflow that will occur during this delay must be taken into consideration in the design of the storage volume. If the pumping rate exceeds the rate of inflow, the water level will drop until it reaches the first pump stop elevation. With the pump stopped, the basin begins to refill and the cycle is repeated. This scenario illustrates that the cycling time will be lengthened by increasing the amount of storage between pump on and off elevations. This volume of storage between first pump on and off elevations is termed usable volume. In theory, the minimum cycle time allowable to reduce wear on the pumps will occur when the inflow to the usable storage volume is one-half the pump capacity.

There are two basic cycling sequences. One is referred to as the "common off elevation." In this sequence, the pumps start at successively higher elevations as required; however, they all stop at the same off elevation. This is advantageous when large amounts of sediment are anticipated. The other sequence uses a "successive start/stop" arrangement in which the start elevation for one pump is also the stop elevation for the subsequent pump, i.e., the start elevation for pump 1 is the stop elevation for pump 2, the start elevation for pump 2 is the stop elevation for pump 3, etc. (see Figure 14-7). There are countless variations between these two sequences.

14.4 Design Concepts and Criteria (continued)

14.4.3.1 Cycling Sequence And Volumes (continued)

There are also different pump start alternation techniques that reduce the cycling volume requirement and equalize wear on the pumps. They range from simply alternating the first pump to start, to continuously alternating all pumps during operation, a technique referred to as "cyclical running alternation". Using this technique, each pump is stopped in the same order in which it starts, i.e., the first pump to start will be the first pump to stop, etc. (see Figure 14-8).

Alternating the first pump to start is sufficient for stormwater pump stations where more than one pump on will be rare and of short duration. This alternation technique coupled with the successive start/stop cycling sequence requires the smallest total cycling volume possible (see Figure 14-9). This total volume is computed as follows:

$$\mathbf{V_t = Q_p\, t_c/4N} \qquad \mathbf{(14.2)}$$

where: V_t = total cycling storage volume, ft^3
Q_p = total capacity of all pumps, ft^3/sec
t_c = minimum allowable cycle time, sec (= 3600/max. starts per hour)
N = total number of equal-size pumps

The proof is as follows:
t = Time between starts
t = Time to Empty + Time to Fill usable storage volume V_t

$$\mathbf{t = V_t/(Q_p - Q_i) + V_t/Q_i} \qquad \textbf{When } \mathbf{Q_i = Q_p/2,\ ft^3/sec,}$$

$$\mathbf{t = 4V_t/Q_p,\ s} \qquad \mathbf{(14.4)}$$

or for t in minutes $t = 4V_t/60Q_p = V_t/15Q_p$

Pump Alteration Sequence Effect

Let us take an example when four pumps are installed in the same station with pumps starting in sequence and stopping in reverse order.

- By designing the control system for pump alternation, sump volumes will be reduced as well as distribute the pump operating time more evenly between the four pumps.
- If the inflow is less than the capacity of one pump, pump number one would, without alternation, do all the work.
- With alternation pump number one starts and draws down. Next start would call pump number two, etc.
- This means that with four pumps of the same size and operating in an alternating sequence each pump is called on to pump down sump volume V_1 every fourth time. The cycle time of each pump will be four times longer than the cycle time of filling and emptying of V_1.

14.4 Design Concepts and Criteria (continued)

14.4.3.1 Cycling Sequence And Volumes (continued)

- The volume required for each pump will vary, depending upon the characteristics of the discharge system. It should be noted that with these volumes, the minimum allowable cycle time will only be experienced when the proportionate inflow to each pump is exactly one-half the capacity of that pump. All other inflows will produce a cycle time longer than the minimum.
- This system works for any number of pumps in a station.

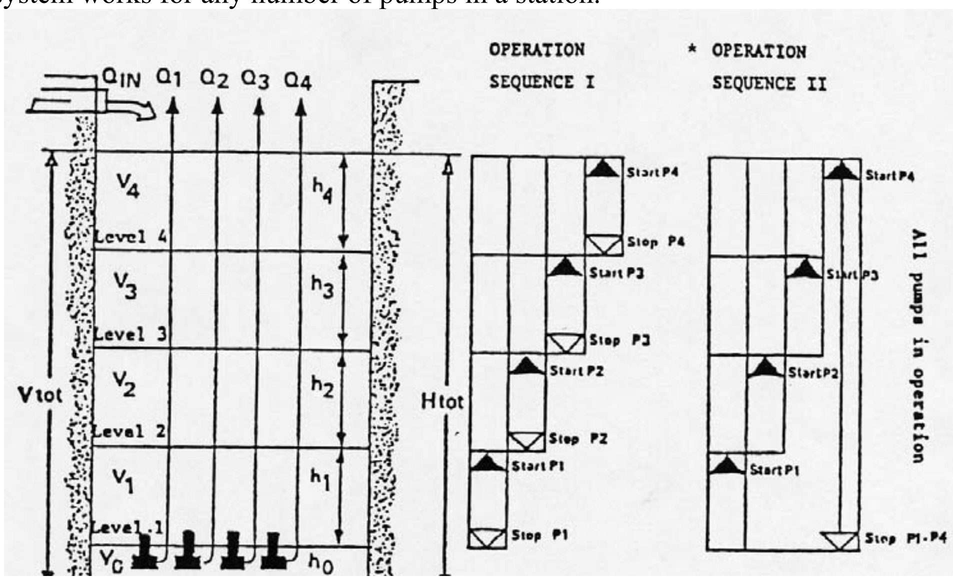


Figure 14-7 Pump Sequences

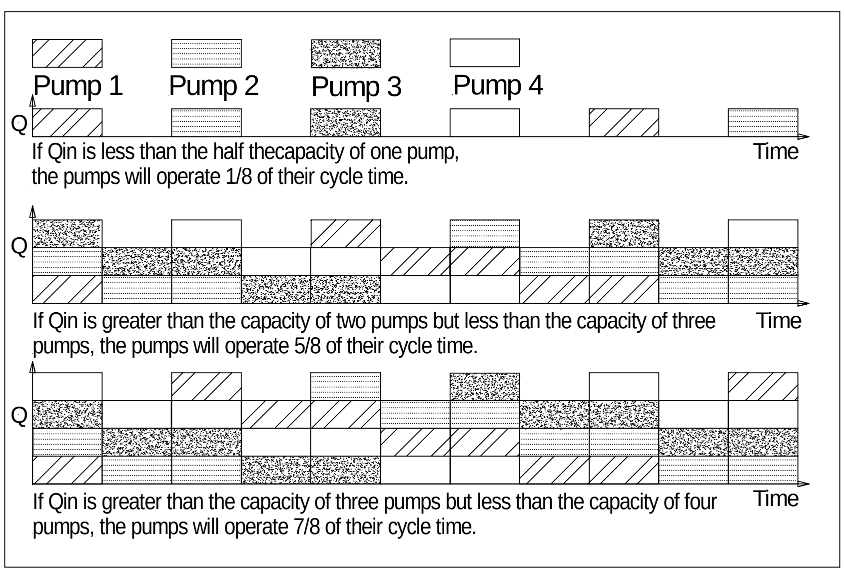
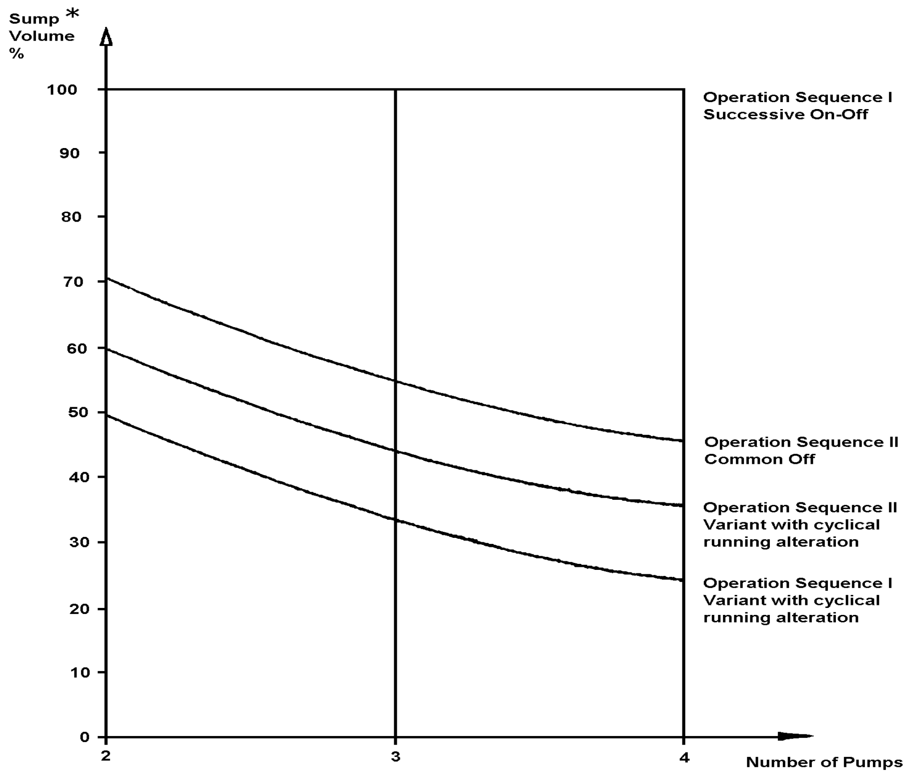


Figure 14-8 Schematic of Pump Sequences at Different Inflow Rates Pumps With Cyclical Running Alternation -- A Variant Of Operation Sequence I in Figure 14-7

14.4 Design Concepts and Criteria (continued)

14.4.3.1 Cycling Sequence And Volumes (continued)



100% corresponds to the volume that is derived from the formula $V_t = (Q_p T_{min})/4N$

Figure 14-9 Comparison Of The Pump Volume with and without Cyclical Running Alternation

Lowest Pump "Off" Elevation

It is recommended that the lowest pump "off" elevation be located at or within 1 foot below the inlet invert elevation unless plan dimension constraints dictate that the station floor be lowered to obtain the necessary cycling volume. This recommendation is based on the fact that it is usually less expensive to expand a station's plan dimensions than to increase its depth. This elevation represents the static pumping head to be used for pumping selection

Pump "On and Off" Elevations

These should be set at the elevations, which satisfy the individual pump cycling volumes (V_x). Starting the pumps as soon as possible by incrementing these volumes successively above the lowest pump off elevation will maximize what storage is available within the wet well and the collection system. The depth required for each volume is computed as follows:

$$\mathbf{H_x = V_x/plan\ area} \qquad \mathbf{(14.3)}$$

14.4 Design Concepts and Criteria (continued)

14.4.3.1 Cycling Sequence And Volumes (continued)

Pumping Range

It is recommended that the minimum distance between the “on” and “off“ elevation of an individual pump be 6 inches. If it is less than 6 inches, then reduce the plan area. Though rare, this could require a reduction in the number of pumps, an increase in station depth, or both.

Allowable High Water Elevation

The allowable high water (AHW) elevation in the station should be set such that the water surface elevation at the lowest inlet in the collection system provides 0.5 feet of freeboard below the roadway grate.

Clearances

Pump to pump, pump to back wall, and pump to sidewall clearances should be the minimum possible to minimize the potential for sedimentation problems. Consult manufacturer literature or a dimensioning guide. The pump inlet to floor clearance plus the pump submergence requirement constitutes the distance from the lowest pump "off" elevation to the wet well floor. The final elevation may have to be adjusted as the type of pump to be installed is finalized.

Intake System Design

The primary function of the intake structure is to supply an even distribution of flow to the pumps. An uneven distribution may cause strong local currents resulting in reduced pump efficiency and undesirable operational characteristics. The ideal approach is a straight channel coming directly into the pump or suction pipe. Turns and obstructions are detrimental, since they may cause eddy currents and tend to initiate deep-cored vortices. The inflow should be perpendicular to a line of pumps and water should not flow past one pump to get to another. Unusual circumstances will require a unique design of the intake structure to provide proper flow to the pumps.

ADOT Freeway Design Practice

Pump on/off elevations for the primary pumps are determined during final design of the station considering the pump characteristic minimum cycle time, and total storage available (both underground plus wet well). The recommended minimum pump cycle time is 20 minutes.

The sump pump is set to turn on two feet below the level of the intake pipe invert and turn off at the wet well floor elevation. The first main pump should turn on at a slightly higher level than the sump pump and both pumps will be on for a portion of the total range. When the wet well level backs into the intake/storage pipes, the volume of storage in the sloping pipe is calculated as described in Appendix A for an “ungula”.

14.4 Design Concepts and Criteria (continued)

14.4.4 Inflow-Outflow Routing

14.4.4.1 Storage

The development of the wet-well design as discussed in Section 14.4.3 has general application when it is anticipated that most of the peak flow will be pumped. In that case, pump run time and cycling sequences are of great importance. In the case of many of the highway storm drain situations, it has been the practice to store substantial parts of the flow in order to minimize pumping requirements as well as outflow piping. The demand on the pumping system is different and thus additional considerations need to be made.

The total storage capacity that can or should be provided is an important initial consideration in pump station design. The designer should recognize that a balance should be reached between pump rate and storage volume. This will require a trial and error procedure used in conjunction with an economic analysis. Using the hydrograph and pump-system curves, various levels of pump capacity can be tried and the corresponding required total storage can be determined. The basic principle is that the volume of water as represented by the shaded area of the hydrograph in Figure 14-10 is beyond the capacity of the pumps and must be stored. If a larger part or most of the design storm is allowed to collect in a storage facility, a much smaller pump station can be utilized, with anticipated cost benefits. The principles discussed for minimum run time, pump cycling, etc. in the design of wet wells should also be considered in the case of larger storage volume development. However, it will be noted that differences exist as the volume of storage become larger. Typically, the concern for meeting minimum run times and cycling time will be reduced because the volume of storage is sufficient to prevent these conditions from controlling the pump operation. The start and stop elevations will be of different magnitudes because of the volume represented by each increment of storage depth.

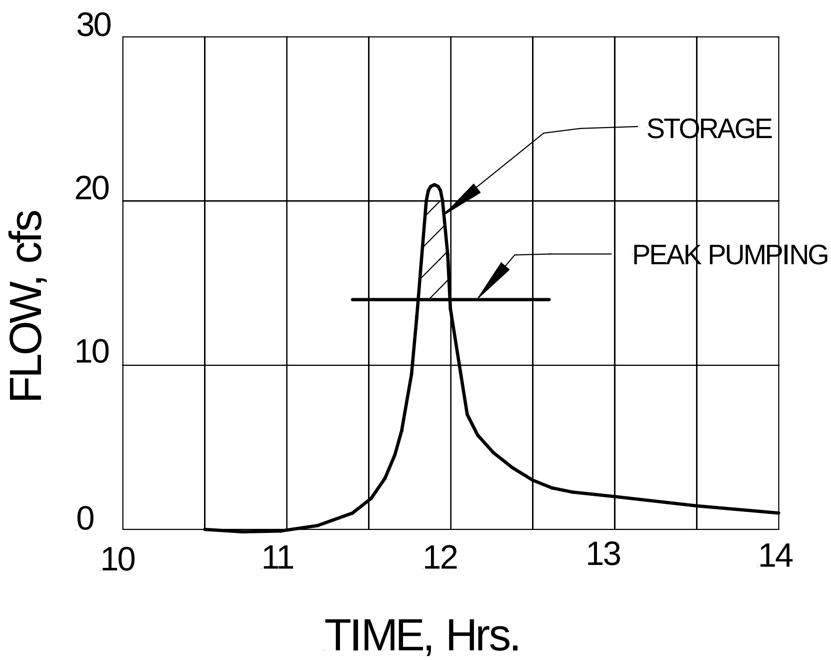


Figure 14-10 Estimating Required Storage

14.4 Design Concepts and Criteria (continued)

14.4.4 Inflow-Outflow Routing (continued)

14.4.4.2 Mass Curve Diagram

The approach used for the design of the pump station is that associated with the development of an inflow mass curve, figure 14-11. The mass inflow curve procedure is commonly used when significant storage is provided outside of the wet well. The plotting of the performance curve on the mass inflow diagram gives the designer a good graphical tool for determining storage requirements. The procedure also makes it easy to visualize pump start/stop and run times. In the event that a pump failure should occur, the designer can also evaluate the storage requirement and thus the flooding or inundation that could occur. The graphical procedure described below is approximate and labor intensive. Computer solutions of the method should be used for the analysis of pumping systems.

In this process, the designer uses an inflow hydrograph and a developed stage-storage relationship. Trial pumping systems will be applied to the inflow mass curve to develop a mass curve routing diagram. The inflow hydrograph is a fixed design component while the storage and pumping discharge rates are variable. The designer will select a design pumping discharge rate, this may be based on downstream capacity considerations. With the inflow mass curve and an assigned pumping rate, the required storage volume can be determined by various trials of the routing procedure.

As the stormwater flows into the storage basin, it will accumulate until the first pump start elevation is reached. The first pump is activated and if the inflow rate is greater than the pump rate, the stormwater will continue to accumulate until the second pump start elevation is reached. As the inflow rate decreases, the pumps will shut off at their respective pump stop elevations. These conditions are modeled in the mass curve diagram by establishing the point at which the cumulative flow curve has reached the storage volume associated with the first pump-start elevation. This storage volume is represented by the vertical distance between the cumulative flow curve and the base line. A vertical storage line is drawn at this point since it establishes the time at which the pump first starts. The pump discharge line is drawn from the intersection of the vertical storage line and the base line upwards toward the right; the slope of this line is equal to the discharge rate of the pump. The pump discharge curve represents the cumulative discharge from the storage basin, while the vertical distance between the inflow mass curve and the pump discharge curve represents the amount of stormwater stored in the basin.

If the rate of inflow is greater than the pump capacity, the inflow mass curve and the pump discharge curve will continue to diverge until the volume of water in storage is equal to the storage associated with the second pump-start elevation. At this point the second pump starts, and the slope of the pump discharge line is increased to equal the combined pumping rates. The procedure continues until peak storage conditions are reached. At some point on the inflow mass curve, the inflow rate will decrease, and the slope of the inflow mass curve will flatten. To determine the maximum amount of storage required, a line is drawn parallel to the pump discharge curve and tangent to the inflow mass curve. The vertical distance between the lines represents the maximum amount of storage required.

The routing procedure continues until the pump discharge curve intersects the inflow mass curve. At this point the storage basin has been completely emptied, and a pumping cycle has been completed. As the storm recedes, the pumps will cycle to discharge the remaining runoff.

14.4 Design Concepts and Criteria (continued)

14.4.4.2 Mass Curve Diagram (continued)

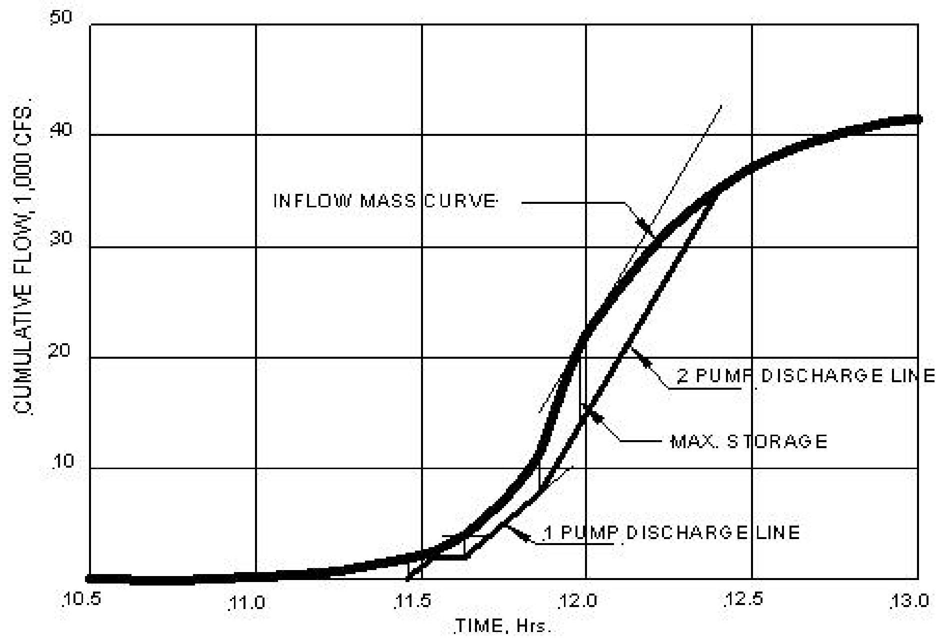


Figure 14-11 Mass Curve Diagram

In developing the pump discharge curve, the designer should remember that the pump's performance curve is quite sensitive to changes in head and that the static head will fluctuate as the water level in the storage basin fluctuates. The designer should also recognize that the pump discharge rate represents an average pumping rate.

14.5 Design Procedure

14.5.1 Introduction

The following is a systematic procedure that integrates the hydraulic design variables involved in sump design. It incorporates the above recommended design criteria and yields the required number and capacity of pumps as well as the wet well and storage dimensions. The final dimensions can be adjusted as required to accommodate non-hydraulic considerations such as maintenance.

Theoretically an infinite number of designs are possible for a given site. Therefore, to initiate design, constraints must be evaluated and a trial design formulated to meet these constraints. Then by routing the inflow hydrograph through the trial pump station its adequacy can be evaluated.

Design Procedure:

1.) Develop Inflow Hydrograph
2.) Estimate Pumping Rate, Volume of Storage, and Number of Pumps
3.) Determine High Water Level
4.) Determine Pump Pit dimensions,
5.) Pump Cycling and Usable Storage
6.) Estimate Volume of Storage
7.) Stage-Storage Relationship
8.) Determine Total Dynamic Head, Net Positive Head, and Head Capacity Curves
9.) Pump Design Point
10.) Power Requirements
11.) Mass Curve Routing
12.) Documentation

Design Checklist

Initial Data

Contributing Drainage Area
Locating of Outfall
Capacity of Outfall
HYDROLOGY
Environmental Considerations

Possible Components

Building Architectural

Location
Capacity
Storage
Pit Type
Equipment Access
Hoisting Equipment
Safety
Ventilation Equipment
Potable Water Supply
Hazardous Materiel Containment

14.5 Design Procedure (continued)

Design Checklist (continued)

Possible Components

Mechanical System
Pump Type
Power Source
Monitor & Control Systems
Sediment Handling

Collection System
- Inlets
- Trash Rack
- Grit Chamber

Discharge System
Piping
Valving

Hydraulic Analysis
Pump Characteristics
Pipe Losses
Miscellaneous Losses
Mass Curve Routing

14.5 Design Procedure (continued)

14.5.2 Pump Station Design

The procedure for pump station design is illustrated in the following 11 steps.

Step 1 Inflow to Pump Station

Develop inflow hydrograph to the pump station using the procedures presented in the ADOT Hydrology Manual. For highway pump stations where the inflow is of short duration and high intensity, a hydrograph that correctly depicts the time/area/discharge relationship should be used, even for small drainage areas. For small basins, HEC-1 can be used to develop the hydrograph in a two stage process. See Appendix B in Chapter 15.

Step 2 Estimate Maximum Discharge, Pumping Rate and Number of Pumps

Because of the complex relationship between the variables of pumping rates, storage and pump on-off settings, a trial and error approach is usually necessary for estimating the pumping rates and storage required for a balanced design. A wide range of combinations will produce an adequate design. The goal is to develop an economic balance between volume and pumping capacity.

Determine the required type, size, and capacity of pumps and power unit characteristics. Usually the minimum number is 3, however this may be 2 for small pump stations or more than 3 if each pump is limited by power unit size. Develop the system curve for the proposed discharge system and superimpose it on the pump performance curves.

Determine the maximum number of starts/hour (minimum cycle time) for the type of power unit proposed.

Step 3 Allowable High Water Level

The highest permissible water level must be set as 0.5 to 2 feet below the finished pavement surface at the lowest pavement inlet.

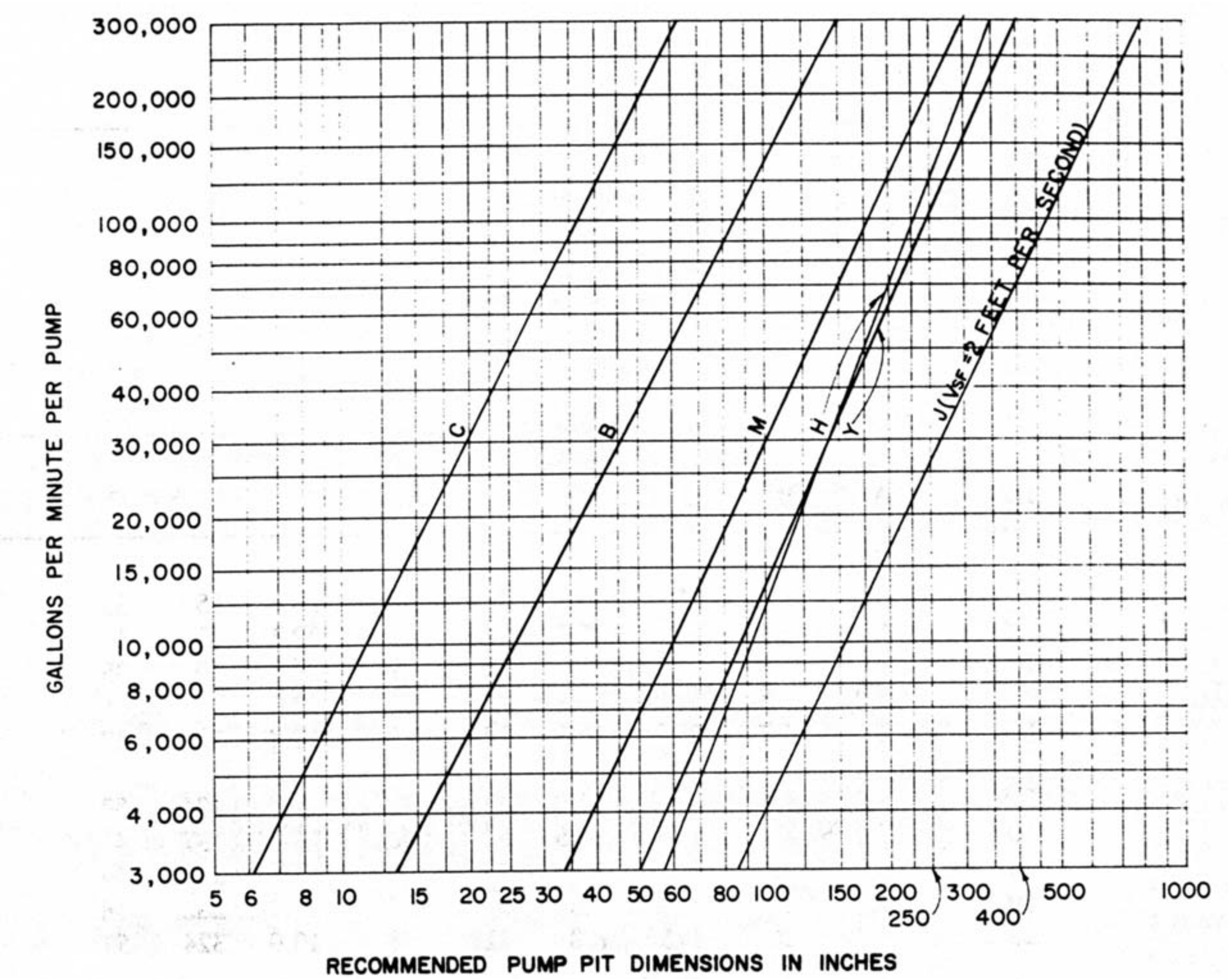
At the design inflow, some head loss will occur through the pipes and appurtenances leading to the pump station. Therefore a hydraulic gradient will be established and the maximum permissible water elevation at the station will be the elevation of the hydraulic gradient. This gradient will be very flat for most wet well designs with exterior storage because of the unrestricted flow into the wet well. Determine the inflow invert elevation

Step 4 Determine Pump Pit Dimensions

Determine the minimum required plan dimensions (LxW) for the pump station from manufacturer's literature or from dimensioning guides such as those provided by the Hydraulic Institute, see Figures 14-12 and 14-13. The dimensions are usually determined by locating the selected number of pumps on a floor plan keeping in mind the guidance given in Section 14.4.3 for clearances and intake system design. Keep in mind the need for clearances around electrical panels and other associated equipment that will be housed in the pump station building.

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)



J = Minimum distance from trash rack to backwall (length of Pump Pit).
V_{SF}= Maximum Velocity of stream flow. (0.5 ft./sec. recommended)
B = Maximum distance from pump centerline to backwall.
C = Average distance from underside of bell to bottom of pit
H = Minimum distance from minimum water level to bottom of pit.
M = Minimum center-to-center of pumps.
Y = Minimum distance to pump centerline from downstream end of any obstruction in sump. (Obstruction must be streamlined.)

Figure 14-12 Recommended Rectangular Pump Pit Dimensions

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

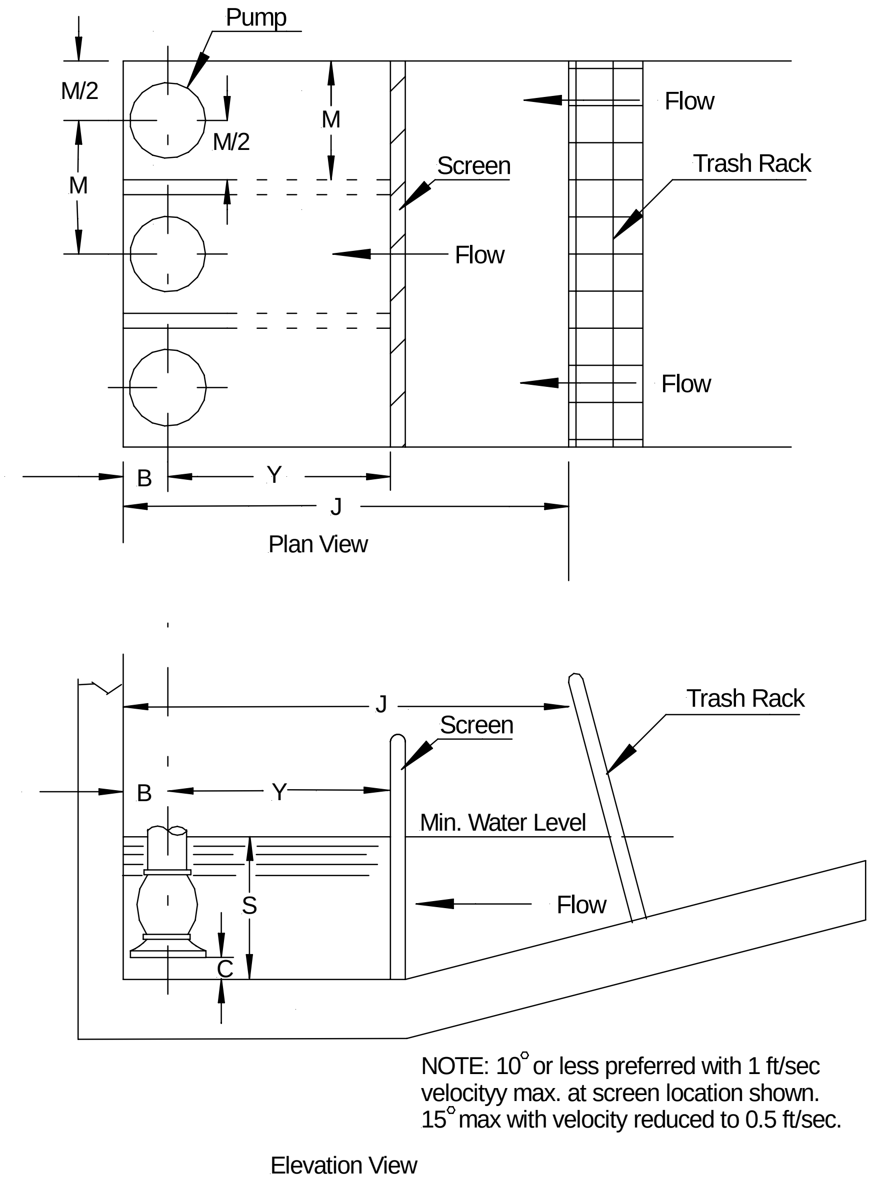


Figure 14-13 Sump Dimension, Wet Pit Type Pumps

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 5 Pump Cycling and Usable Storage

One of the basic parameters addressed initially was that the proper number of pumps must be selected to deliver the design Q. Also, the correct elevations must be chosen to turn each pump on and off. Otherwise, rapid cycling (frequent starting and stopping of pumps) may occur causing undue wear and possible damage to the pumps. The volume of storage between first pump on and off elevations is termed usable volume. For a given pump with a capacity Q_p, cycling will be a maximum (least time between starts) when the inflow Q_i to the usable storage is one-half the pump capacity. Assuming this condition, usable volume can be related to cycling time

Generally, the minimum allowable cycling time, t, for electrically driven pumps is designated by the pump manufacturer based on electric motor size. In general, the larger the motor, the larger is the starting current required, the larger the damaging heating effect and the greater the cycling time required. The pump manufacturer should always be consulted for allowable cycling time during the final design phase of project development.

However, the following limits may be used for estimating allowable cycle time during preliminary design:

Motor kW	Cycling Time (t), min
0 - 11	5.0
15 - 22	6.5
26 - 45	8.0
48 - 75	10.0
112 - 149	13.0

Knowing the pumping rate and minimum cycling time, the minimum necessary allowable storage, V, to achieve this time can be calculated by:

$$\mathbf{V = 15\, Q_p t} \qquad \textbf{(14.5)}$$

Where:

Q_p = pump capacity, cfs.
t = time between starts, in minutes.

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 5 Pump Cycling and Usable Storage (continued)

When larger volumes of storage are available, the initial pump start elevations can be selected from the stage-storage curve. Since the first pump turned on should typically have the ability to empty the storage facility, its turn off elevation would be the bottom of the storage basin. The elevation associated with the minimum allowable storage volume in the stage-storage curve is the lowest turn-on elevation that should be allowed for the starting point of the first pump. The minimum allowable storage is calculated by the equation $V=15\ Q_pt$.

Having selected the trial wet-pit dimensions, the pumping range, Δh, can then be calculated. The pumping range represents the vertical height between pump start and pump stop elevations. Usually, the first pump stop elevation is controlled by the minimum recommended bell submergence criteria specified by the pump manufacturer or the minimum water level, H, specified in the design. The first pump start elevation will be a distance, Δh, above H. When the only storage provided is in the wet pit, the pumping range can be calculated by dividing the required storage volume by the wet pit area.

$$\Delta h = V/\text{wet pit area} \tag{14.6}$$

The minimum plan area of the wet pit based on the pump layout is checked by comparison of the required pumping range with the available pumping range. Determine the design high water elevations (DHW) by adding the storage depth, Δh, to inlet invert elevation. DHW=INV+ Δh. If DHW is less than or equal to the allowable high water elevation (AHW), the dimensions are satisfactory. If the DHW is greater than the AHW, compute

$$X=Vt/[L(AHW-INV)] - W$$

Where L= length of wet well, dimension perpendicular to the line of pumps)
W= width of wet well

If X is less than the width required between pumps compute the new wet well length as

$$L=Vt/[(W(AHW-DHW)],$$ The wet well width should not be increased.

If X is greater than the width required between pumps, and one pump and return to step 2 (i.e., determine the new pump/motor characteristics, wet well dimensions, and cycle time storage volumes).

The second and subsequent pump start elevations will be determined by plotting the pump performance on the mass inflow curve. This distance between pump starts may be in the range of 1 to 3 feet for stations with a small amount of storage and 0.25 to 0.5 feet for larger storage situations.

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 6 Estimate Volume of Storage

Some approximation of storage in addition to the pumping rate is necessary to produce the first trial design. Using the approach presented in section 14.4.4.1 the peak pumping rate is assigned and a horizontal line representing the peak rate is drawn across the top of the hydrograph. The shaded area above the peak pumping rate represents an estimated volume of storage required above the last pump turn on point. This area is measured to give an estimated starting size for the storage facility. Once an estimated storage volume is determined, a storage facility can be estimated. The shape, size, depth, etc., can be established to match the site and a stage-storage relationship can be developed.

Step 7 Stage-Storage Relationship

Routing procedures require that a stage-storage relationship be developed. This is accomplished by calculating the available volume of water for storage at uniform vertical intervals.

Having roughly estimated the volume of storage required and trial pumping rate by the approximate methods described in the preceding sections, the configuration and elevations of the storage chamber can be initially set. Knowing this geometry, the volume of water stored can be calculated for its respective depth. In addition to the wet-pit, storage will also be provided by the inflow pipes and exterior storage if the elevation of water in the wet-pit is above the inflow invert. If the storage pipe is circular, the volume can be calculated using the ungula of a cone formula as discussed in Appendix A, Figure 14-A1. Figures 14-A1, 14-A2 and 14-A3 give examples of the calculation and plotting of the storage in a circular pipe and a circular wet pit. A similar procedure would be followed for other storage configurations. Volume in a storage chamber can be calculated below various elevations by formulas depending on the shape of the chamber. A storage vs. elevation curve can then be plotted and storage below any elevation can readily be obtained.

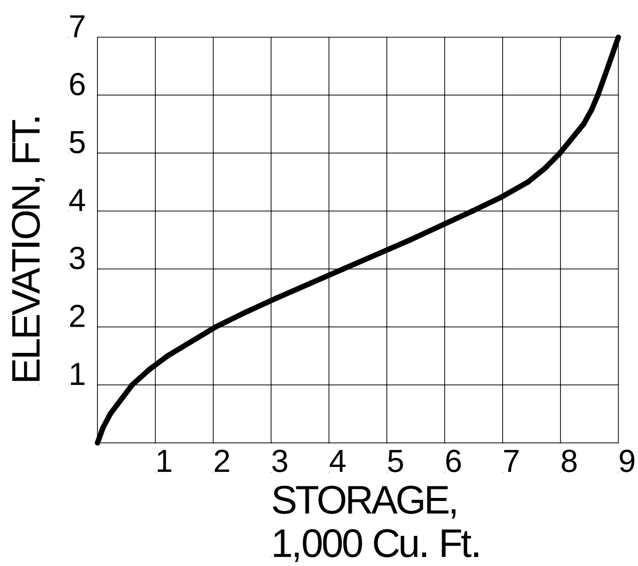


Figure 14-14 Stage-Storage Curve

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 8 Total Dynamic Head

Total Dynamic Head is the sum of the static head, velocity head and various head losses in the pump discharge system due to friction. Knowing the range of water levels in the storage pit and having a trial pump pit design with discharge pipe lengths and diameters and appurtenances such as elbows and valves designated, total dynamic head for the discharge system can be calculated.

The designer must select a specific pump in order to establish the size of the discharge piping that will be needed. This is done by using information either previously developed or established. Though the designer will not typically specify the manufacturer or a specific pump, study of various manufacturers' literature will assist in establishing reasonable relationships between total dynamic head, discharge, efficiency and energy requirements. This study will also give the designer a good indication of discharge piping needed since pumps that produce the desired results will have a specific discharge pipe size.

To summarize the Total Dynamic Head (TDH) is equal to: $TDH = H_s + H_f + H_v + H_p$

Where: H_s = static head or height through which the water must be raised, ft
H_f = loss due to friction in the pipe, ft
H_v = velocity head, ft
H_p = loss due to friction in water passing through the pump valves, fittings and other items, ft.

The Manning's formula expressed as follows is generally used for discharge lines.

$$\mathbf{H_f = L * S_f = L * [(Q * n)/(1.486 * A * R^{2/3})]^2} \quad \textbf{(14.7)}$$

Where: Q = discharge, ft^3/sec
L = length of pipe, ft
n = Manning's roughness value
A = cross sectional area of discharge pipe, ft^2
R = hydraulic radius of discharge pipe, ft (For pipe running full, R = diameter/4)

Friction losses can also be computed by the Darcy Formula. This requires computation of the relative roughness of the pipe, the Reynold's number and the friction factor. The Hydraulic Institute and others have produced line loss tables and charts that make determination of losses quite easy and accurate. The tables and charts have been developed for a variety of pipe materials and are recommended for use in determining line and fitting losses for the discharge side of the pumping system.

Headlosses for other components should be evaluated using the information is the Storm Drainage System Chapter. Standard textbooks and manufacturers' catalogs should also be consulted.

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 9 Pump Design Point

Using methods described in the previous step, the Total Dynamic Head of the outlet system is calculated for a specific static head and various discharges. These TDHs are then plotted vs. discharge. This plot is called a system head curve. A system head curve is a graphical representation of total dynamic head plotted against discharge Q for the entire pumping and discharge system. The required design point of a pump can be established after the pump curve is superimposed to give a visual representation of both system and pump. As usually drawn, the system head curve starts from a low point on the Y-ordinate representing the static head at zero discharge. It then rises to the right as the discharge and the friction losses increase. A design point can be selected on the system head curve and a pump can be selected to match that point. The usual pump curve is the reverse of the system head curve so the point of intersection is clearly identifiable. System head curves are often drawn for several different static heads, representing low, design and maximum water levels in the sump.

One, two or more pump curves can be plotted over the system head curves and conditions examined. If a change of discharge line size is contemplated, a new system head curve for the changed size (and changed head loss) is easily constructed. Figure 14-15 shows the difference in the pump operating range when two pumps are connected to a common discharge line versus separate discharge lines. With a common discharge line the design point will move from A, for the first pump operating alone, to B, with both pumps operating at the minimum static head. Where as, the same two pumps with separate discharge lines operate over a more limited range since the additional loss from a shared pipe is not experienced. In highway design, it is common practice to provide individual discharge lines for each pump. Therefore, the additional loss from a shared pipe is not experienced. It should be noted that the pump will always operate at the intersection of the system curve and the pump curve.

Each pump considered will have a unique performance curve that has been developed by the manufacturer. More precisely, a family of curves is shown for each pump, because any pump can be fitted with various size impellers. These performance curves are the basis for the pump curve plotted in the system head curves discussed above. The designer must study pump performance curves. The designer must have specific information on the pumps available in order to be able to specify pumps needed for the pump station.

Any point on an individual performance curve identifies the performance of a pump for a specific Total Dynamic Head (TDH) that exists in the system. It also identifies the power required and the efficiency of operation of the pump. It can be seen that for either an increase or decrease in TDH, the efficiency is reduced as the performance moves away from the mid-point of the performance curve. It should also be noted that as the TDH increases, the power requirement also increases. The designer must make certain that the motor specified is adequate over the full range of TDHs that will exist. It is desirable that the design point be as close to the mid-point as possible, or else to the left of the mid-point rather than to the right of or above it. The range of the pump performance should not extend into the areas where substantially reduced efficiencies exist.

It is necessary that the designer correlate the design point discussed above with an elevation at about the mid-point of the pumping range. By doing this, the pump will work both above and below the TDH for the design point and will thus operate in the best efficiency range.

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

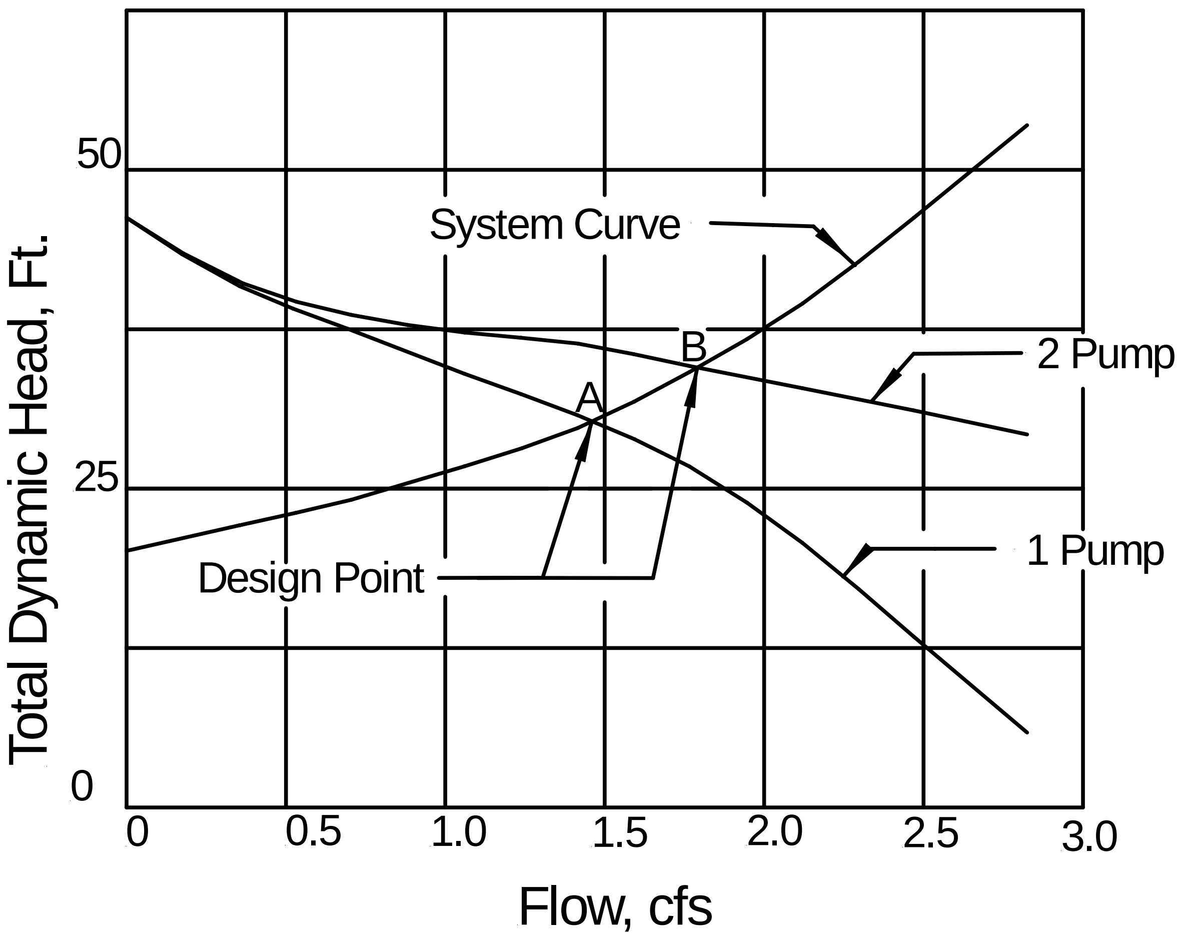


Figure 14-15 System Head Curves

14.5 Design Procedure (continued)

14.5.2 Pump Station Design (continued)

Step 10 Power Requirements

To select the proper size of pump motor, compute the energy required to raise the water from its lowest level in the pump pit to its point of discharge. This is best described by analyzing pump efficiency. Pump efficiency is defined as the ratio of pump energy output to the energy input applied to the pump. The energy input to the pump is the same as the driver's output and is called brake kilowatts.

$$e = Q\gamma H/1000 \text{ brake kW} \tag{14.8}$$

Where: e = efficiency = pump output/brake kilowatts
Q = pump capacity, ft^3/sec
γ = specific weight of liquid (62.4 lbs/ft^3 for cold water)
H = head, ft

Efficiency can be broken down into partial efficiencies — hydraulic, mechanical, etc. The efficiency as described above, however, is a gross efficiency used for the comparison of centrifugal pumps. The designer should study pump performance curves from several manufacturers to determine appropriate efficiency ranges. A minimum acceptable efficiency should be specified by the designer for each performance point specified.

To compute the energy required to drive a pump, assume that the pump will operate at 80% efficiency. The above equation can then be solved for brake kilowatts.

Step 11 Mass Curve Routing

The procedures described thus far will provide all the necessary dimensions, cycle times, appurtenances, etc. to design the pump station. A flood event can be simulated by routing the design inflow hydrograph through the pump station by methods described in Section 14.4.4.2. In this way, the performance of the pump station can be observed at each hydrograph time increment and pump station design evaluated. Then, if necessary, the design can be "fine-tuned."

14.6 References

Flygt. Cost Savings In Pumping Stations.

Hydraulic Institute -- Engineering Data Book.

Hydraulic Institute -- American National Standard for Vertical Pumps.

Hydraulic Institute -- American National Standard for Pump Intake Design.

Federal Highway Administration, “Highway Stormwater Pump Station Design Manual”, HEC24, 2000.

Appendix A
Volume in Ungula

Appendix A Volume in Ungula

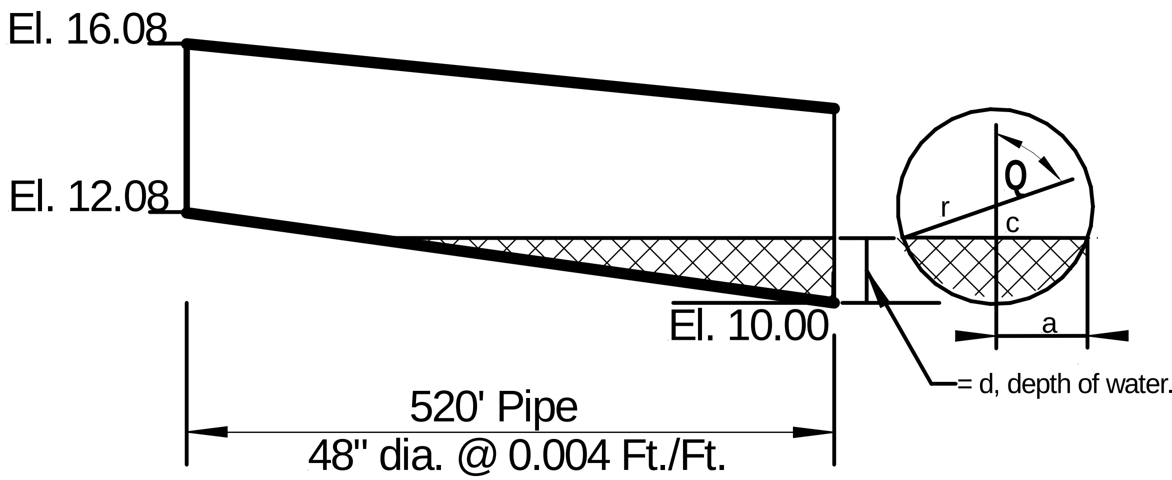


Figure 14-A-1 Storage in Ungula

Source: FHWA IP-82-17, Vol. 1

Ungula Volume: $V = L*(0.67a^3 \pm cB) / (r \pm c)$

Where: L = length of ungula, ft
r = radius of base, ft
B = area of base, ft^2
d = depth of flow, ft
c = distance from water surface to center of circle = r-d = r*sin(Q)
a = ½ width of water surface = $(r^2-c^2)^{0.5}$ = r*cos(Q)

If depth is greater than a semicircle, use + sign. If depth is less than a semicircle, use - sign.

Area of Base, $B = C_aD^2$. See Table A-1, **Area Coefficient -** C_a

Example:
At L = 375 ft, depth = 1.5 ft.; r=2.0 ft;
c = 2.0-1.5 = 0.5; a = $(2.0^2-0.5^2)^{0.5}$ = 1.936
Enter table with d/D = 1.5/4.0 = 0.375. C_a is read from table.
For d/D = 0.375, C_a is interpolated as 0.2691. Therefore B = 0.2691*46 = 4.306 ft^2

The volume for the ungula is $V = L*(0.67a^3 \pm cB) / (r \pm c)$ =
V = $375*(0.67*(1.936)^3-0.5*4.306)/(2.0-0.5) = 375*(4.862-2.153)/(1.5) = 677\ ft^3$

Appendix A Volume in Ungula

Ungula Example

At a depth of 3.5 feet at the entrance, it takes 875 feet to get to zero depth, the depth at the end of the pipe is 3.5 –.004*520 = 1.42.

Therefore volume is the volume for 3.5 feet over 875 feet minus 1.42 feet over 355 feet.

For 3.5 depth, $c = 2.0\text{-}3.5 = -1.5$, $a = (2.0^2-1.5^2)^{0.5} = 1.323$
For $d/D = 3.5/4.0 = 0.875$, C_a is interpolated as 0.729.
Therefore $B = 0.729*16= 11.66$

The volume for the 3.5' ungula is $V = L*(0.67a^3 \pm cB) / (r \pm c) =$
$V = 875*(0.67*(1.323)^3\text{-}(-1.5*11.66)/(2.0\text{-}(-1.5)) = 875*(1.551\text{-}(-17.49)/(3.5) = 4760\ ft^3$

For 1.42 depth, $c = 2.0\text{-}1.42 = 0.58'$, $a = (2.0^2-0.58^2)^{0.5} = 1.914$
For $d/D = 1.42/4.0 = 0.3575$, C_a is interpolated as 0.2498.
Therefore $B = 0.2498*16= 3.997$

Volume for the 1.42 ungula over 355 feet is
$V = 355*(0.67*(1.914)^3\text{-}(0.58*3.997)/(2.0\text{-}(0.58)) = 355*(4.698\text{-}2.318)/(1.42) = 595\ ft^3$

Therefore volume of storage = 4760-595 = 4165 ft^3

At a depth of 5.5 feet, the pipe is full for the first 375 feet. At the end, the depth is 3.42'. The volume is equal to the volume of a full pipe minus the volume of a pipe 0.58 feet.
For a depth of 0.58 feet, the volume is as follows:
$c = 2.0\text{-}0.58 = 1.42$, $a = (2.0^2-1.42^2)^{0.5} = 1.408$

Enter table with $d/D = 0.58/4 = 0.145$, C_a is interpolated as 0.0704.
Therefore $B = 0.0704*16= 1.126\ ft^2$

The volume for the unfilled ungula is $V = L*(0.67a^3 \pm cB) / (r \pm c) =$
$V = 145*(0.67*(1.408)^{3\text{-}}1.42*1.126)/(2.0\text{-}1.42) = 145*(1.870\text{-}1.599)/(0.58) = 68\ ft^3$

Therefore volume of storage = 12.57*520 – 68 = 6467 ft^3

Appendix A Volume in Ungula

Table of Area Coefficient based on ratio of depth of flow to Diameter.

Let D/d = (Depth of water)/(Diameter of pipe) and C_a = the tabulated value.
Then Area = $C_a d^2$

Table A-1
Area Coefficient - C_a

D/d	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0013	0.0037	0.0069	0.0105	0.0147	0.0192	0.0242	0.0294	0.0350
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.0961	0.1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2546	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.393	0.403	0.413	0.423	0.433	0.443	0.453	0.462	0.472	0.482
0.6	0.492	0.502	0.512	0.521	0.531	0.540	0.550	0.559	0.569	0.578
0.7	0.587	0.596	0.605	.0614	0.623	0.632	0.640	0.649	0.657	0.666
0.8	0.647	0.681	0.689	0.697	0.704	0.712	0.719	0.725	0.732	0.738
0.9	0.745	0.750	0.756	0.761	0.766	0.771	0.775	0.779	0.782	0.784

Appendix B
Pump Station Example

Appendix B Pump Station Example

Determine the required storage to reduce the peak flow of 22 cfs to 14 cfs as shown in Figure 14-B-1. Using the assumed storage pipe shown in Figure 14-B-2, the stage-storage curve in Figure 14-B-3, the stage discharge curve in Figure 14-B-4 and the inflow hydrograph in Fig 14-B-1, the storage can be determined.

The inflow mass curve is developed in Figure 14-B-5. Since 14 cfs was to be pumped, it was assumed that two 7 cfs pumps would be used. The pumping conditions are as follows:

	Pump-Start		Pump-Stop	
	Elevation	Volume	Elevation	Volume
Pump No. 1 (7 cfs-3500 gpm)	2.0	2000	0.0	0
Pump No. 2 (7 cfs-3500 gpm)	3.0	4220	1.0	600

The storage volumes (ft^3) are associated with the respective elevations.

To visualize what is happening during the design period, the pump discharge is superimposed on the inflow mass curve as shown in Figure 14-B-6. Note that the first pump is turned on at about hour 11.4 when a storage volume of 2000 ft^3 has accumulated. At about hour 11.5 pump number one has emptied the storage basin and the pump turns off. At about hour 11.7 the storage volume has again reached 2000 ft^3 and a pump is turned on. If an alternating start plan had been developed, this would be the second pump that would turn on at this point. If an alternating start plan had not been designed the first pump would again be started. At about hour 11.8 the volume in storage has increased to 4220 ft^3, which is associated with a turn on elevation of 3.0 ft. Both pumps operate until about hour 12.4 when the volume in the storage basin has been essentially pumped out. The pumps will continue to start and stop until the hydrograph has receded and the inflow stops.

The shaded area between the curves (see Figure 14-B-7) represents stormwater that is going into storage. Pump cycling at the end of the storm has been omitted in order to simplify the illustration. When the stored volume remaining is equal to the volume (600 ft^3) associated with the Pump No. 2 stop elevation (1.0 ft), pump number 2 shuts off. Pump No. 1 shuts off when the storage pipe is emptied at Pump No. 1 stop elevation (0.0).

Appendix B Pump Station Example (Continued)

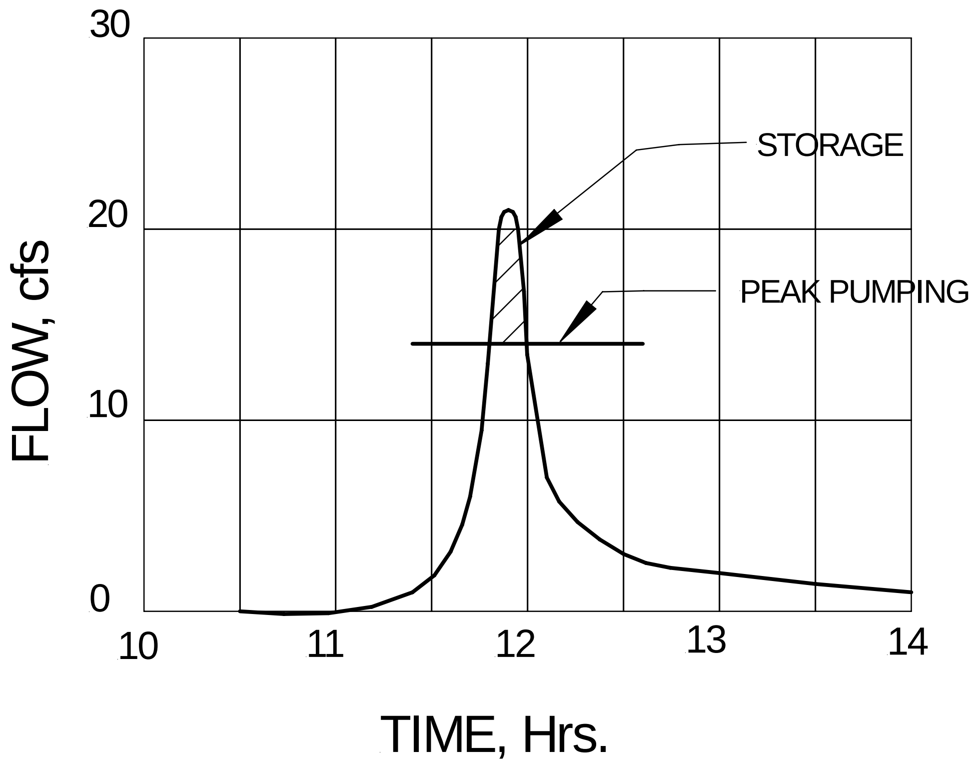


Figure 14-B-1 Example Estimated Required Storage

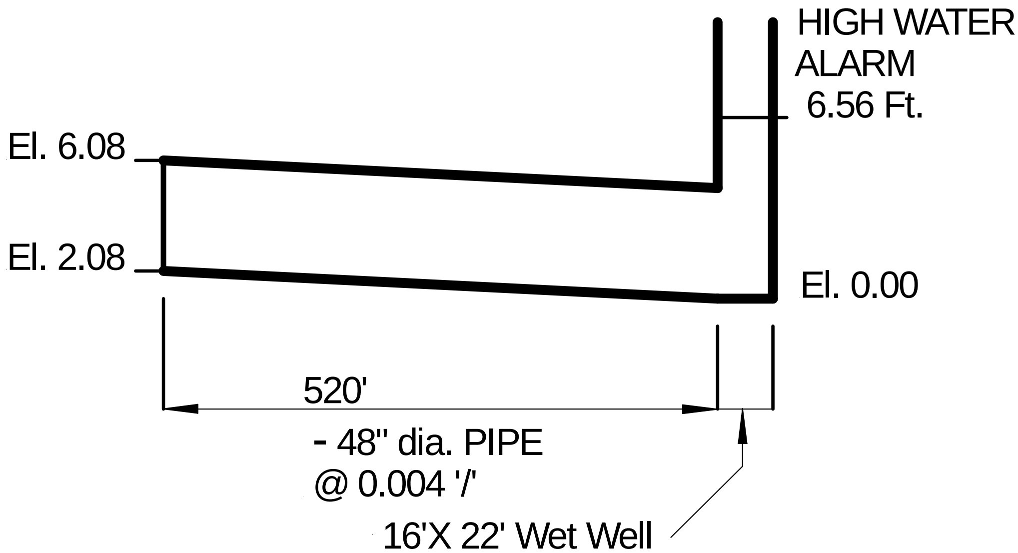


Figure 14-B-2 Storage Pipe Sketch

Appendix B Pump Station Example (Continued)

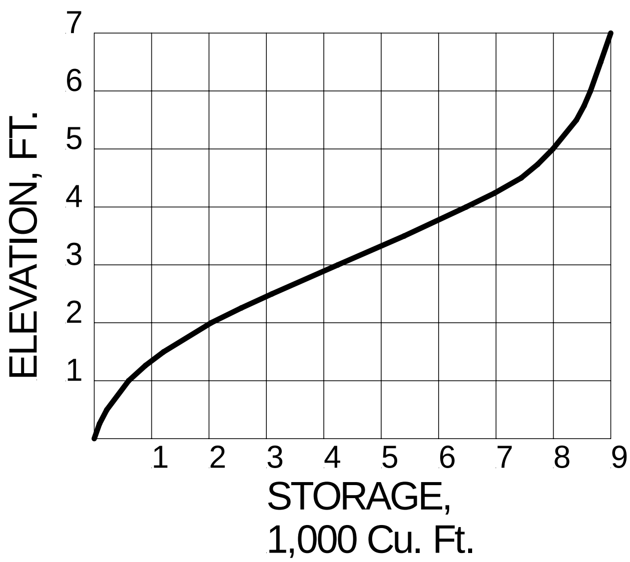


Figure 14-B-3 Stage-Storage Curve

Stage-Storage Tabulation 48 in Pipe at 0.40%, 16' x 22' Wet Well

Elevation (Ft)	Pipe (Ft^3)	Wet Well (Ft^3)	Total (Ft^3)
0.00	0.00	0.00	0.00
0.5	46	176	222
1.0	253	352	605
1.5	673	528	1201
2.0	1334	704	2038
2.5	2213	880	3093
3.0	3188	1056	4244
3.5	4169	1232	5401
4.0	5072	1408	6480
4.5	5772	1584	7356
5.0	6230	1760	7990
5.5	6468	1936	8404
6.0	6534	2112	8646
6.5	6534	2288	8822
7.0	6534	2464	8998

Appendix B Pump Station Example (Continued)

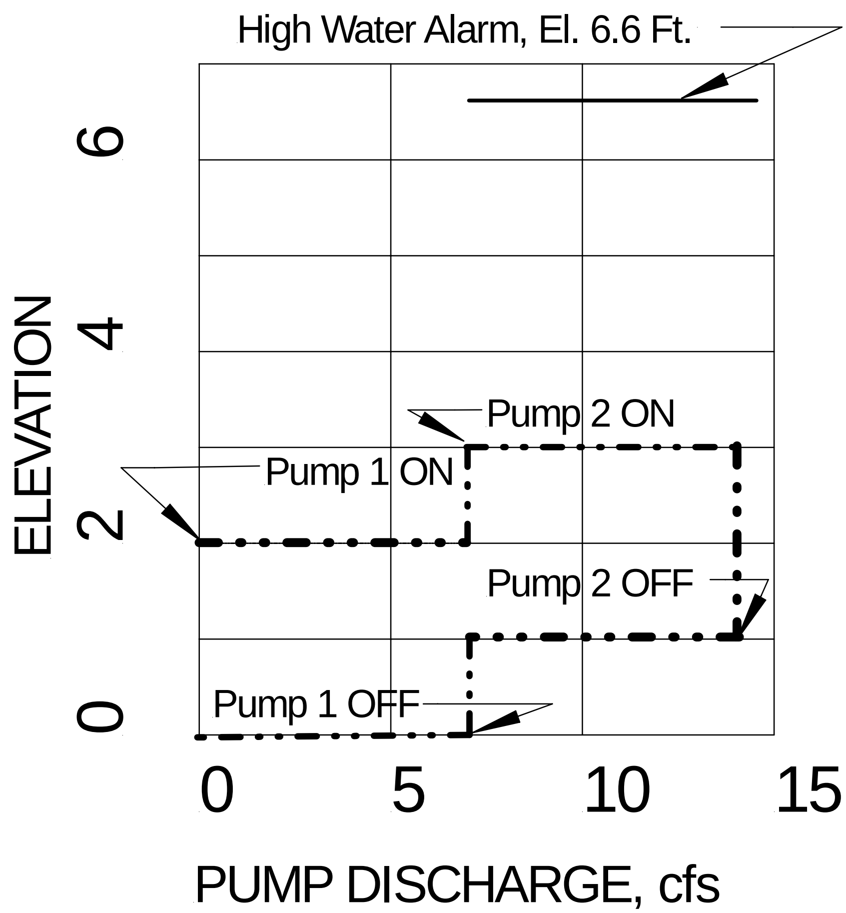


Figure 14-B-4 Pump Stage-Discharge Curve

Appendix B Pump Station Example (Continued)

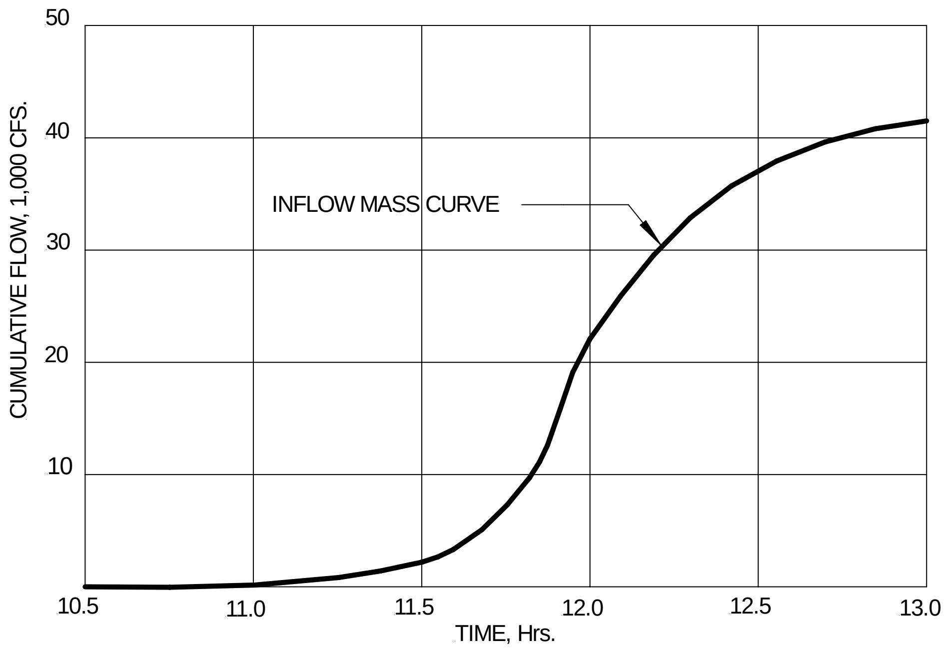


Figure 14-B-5 Development Of Inflow Mass Curve

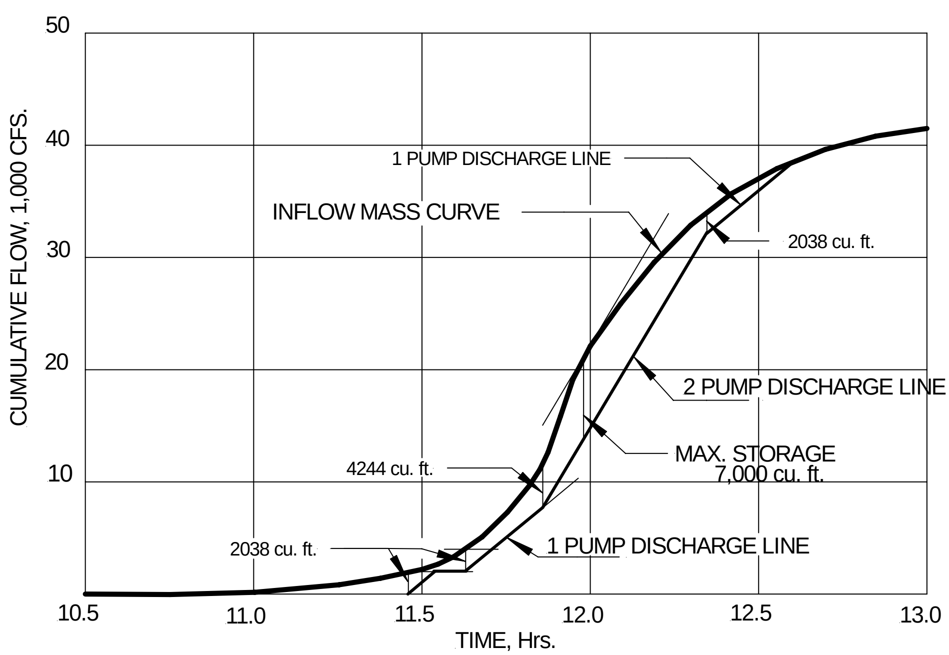


Figure 14-B-6 Mass Curve Diagram

Appendix B Pump Station Example (Continued)

The trial design is now complete. The peak inflow rate of 21 cfs has been reduced to a peak outflow rate of 14 cfs through use of a maximum storage of 8400 ft^3 . A reduction of 33 % has been achieved.

It should be noted that the number of starts per hour can be determined by looking at the plots on the inflow mass curve. Once the mass inflow curve has been developed, it is a relatively easy process to try different pumping rates and different starting elevations until a satisfactory design is developed. This is only one possible design option. Other pumping rates are plotted on the inflow mass curve to determine their performance. Other combinations can be considered. The pumping rate can be reduced by providing more storage. The storage may be reduced by reducing starting elevations.

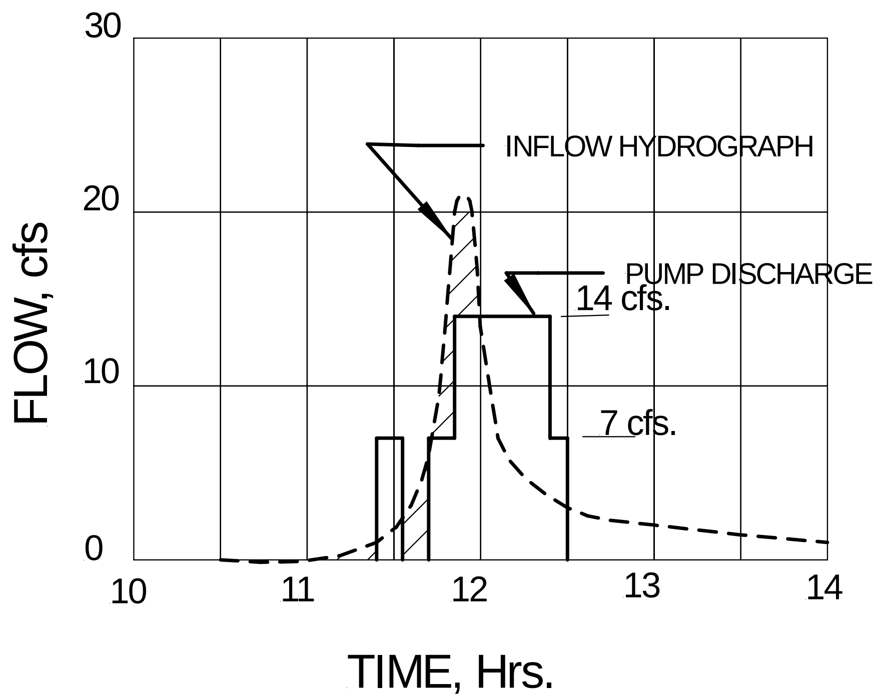


Figure 14-B-7 Pump Operation Hydrographs